Water Power

INTERNATIONAL DEVELOPMENT OF HYDRO-ELECTRIC POWER

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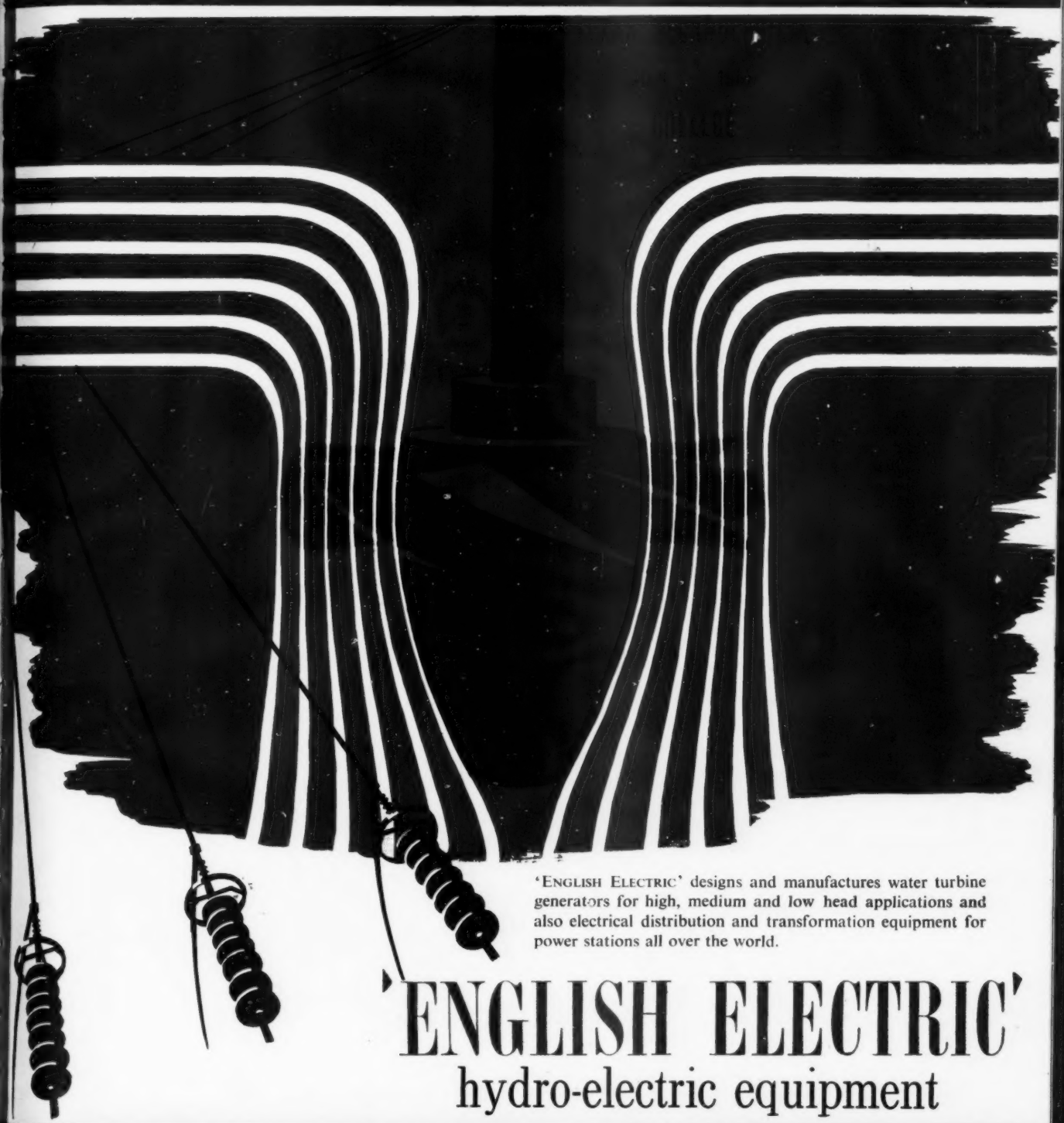

'ENGLISH ELECTRIC' designs and manufactures water turbine generators for high, medium and low head applications and also electrical distribution and transformation equipment for power stations all over the world.

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Equipment for Bradwell

All the switchgear comprising the main 132-kV air-blast circuit-breakers, 6·6-kV, 3·3-kV, and 415-volt air-break equipment for auxiliary supplies, together with its associated control apparatus for Bradwell Nuclear Power Station is being supplied by Reyrolle who are responsible also for the reactor control-rod heads, the fission-products detection equipment, and the machining of the graphite blocks.

Reyrolle specialists in switchgear and control-apparatus

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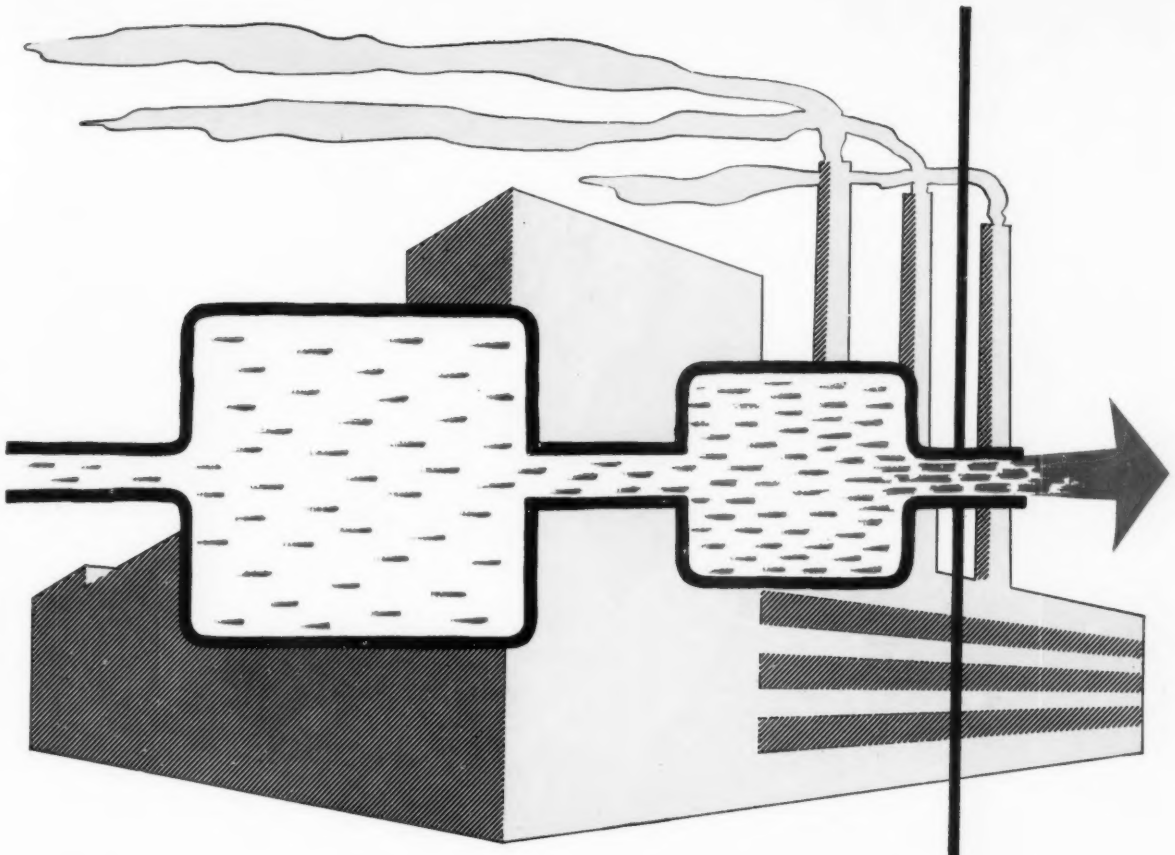

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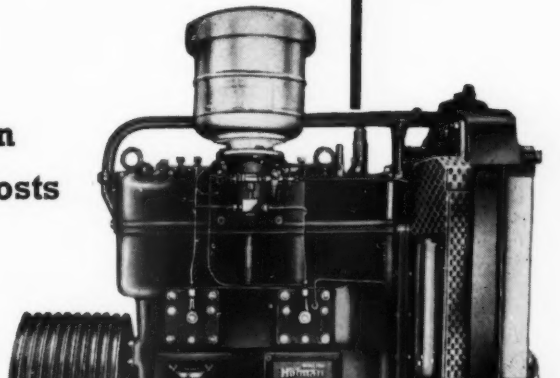

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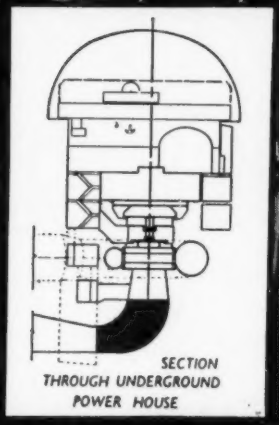
840,000 horsepower from 6 Boving turbines—each of 140,000 h.p.—will be the total output from the first Kariba underground hydro-electric power station. The built-in parts of the first turbine are now being delivered to site on schedule. Erection of the first draft tube liner has already commenced. The size of the scheme can be gauged from the liner, assembled in the works of Babcock & Wilcox, South Africa (PTY) Vereeniging.



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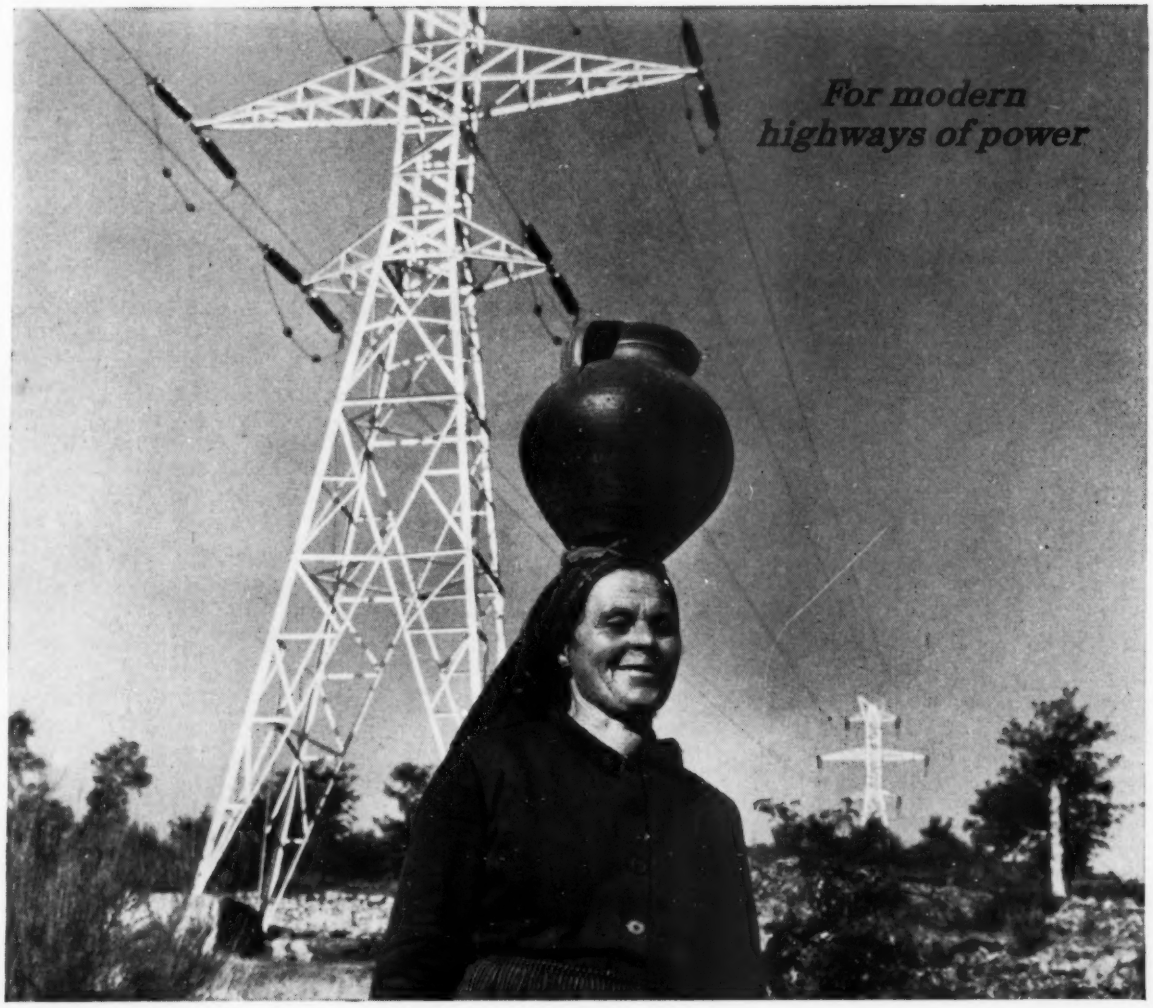
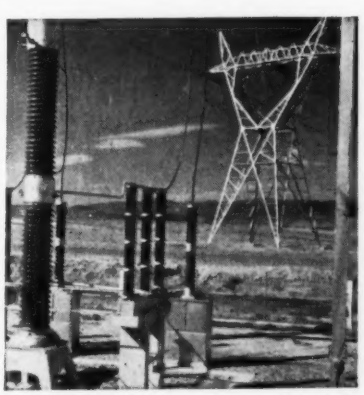


SPAIN CHOOSES FERAL CABLE

In Spain the weather, while in many ways a blessing, is also a problem—tending to extremes of drought, floods, bitter cold, blazing heat. And although it is the driest country in Europe, for power Spain has to depend largely on hydroelectric sources. Fortunately in the mountainous northwestern parts, back of Corunna and Vigo, is an area of considerable rainfall. Here too the narrow valley gorges lend themselves to the creation of storage facilities through dam building. And to carry power from San Esteban, the main dam in the River Sil system, to Madrid and other consuming areas, the Spanish utility company naturally chose FERAL CABLE—*reliable* Swedish ACSR which today is installed in important power lines the world over.



Substation at Majadahonda, end of the 220,000-volt trunk line from San Esteban to Madrid. Here power, transformed down to 132,000 volts, is fed into two lines connecting with the systems of other utilities in the Madrid area. Further substations through which power is supplied to local networks are located at Tordesillas and Trives.

When planning power lines, consult

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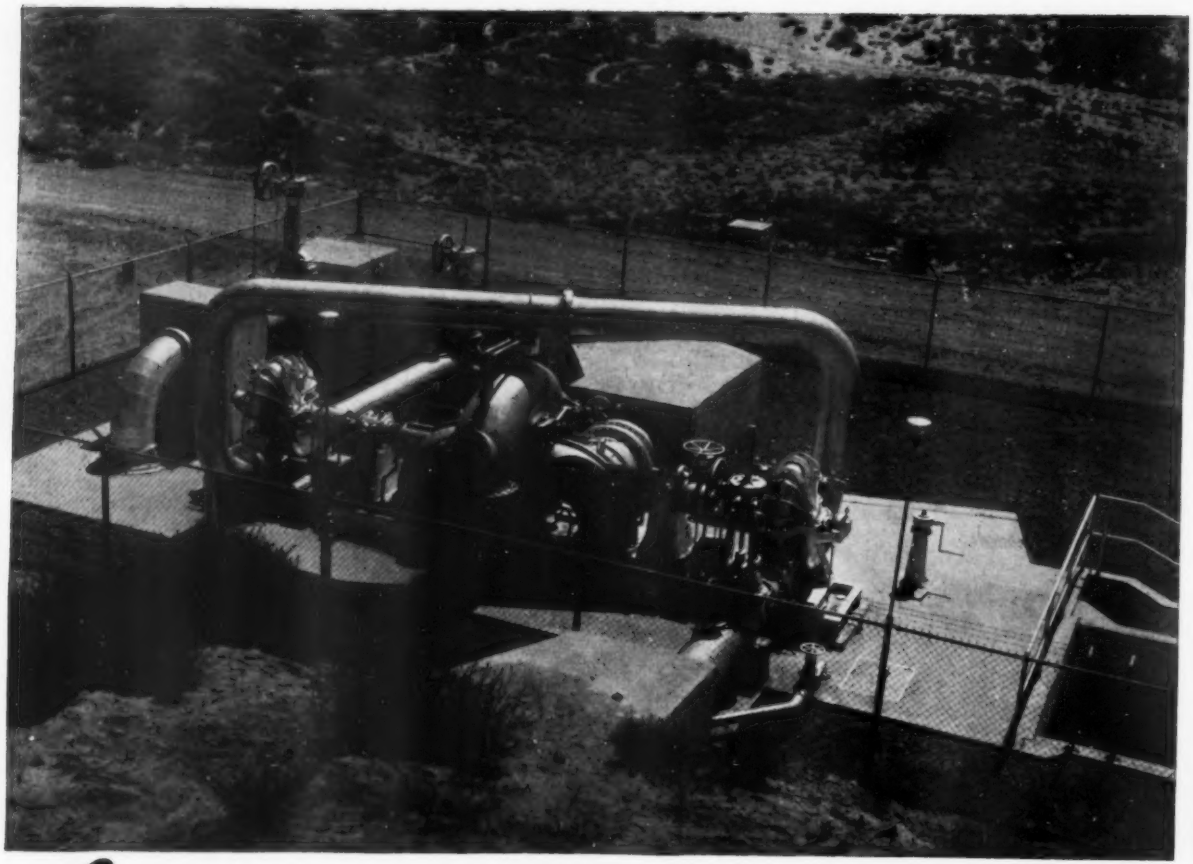

Another LEFFEL TURBINE FOR KENNEWICK

Now, there are Leffel hydraulic turbines at work in two Kennewick Division stations . . . the recently completed Amon Pumping Plant and the Chandler Power and Pumping Plant. Located in the southeastern section of the State of Washington, the Kennewick Division of the Yakima Project is being developed by the Bureau of Reclamation of the U.S. Department of the Interior.

The Amon Pumping Plant, unusual because of its outdoor installation, is powered by a Leffel turbine which withstands the effects of wind and sand, in addition to performing at top turbine efficiency. This turbine is the horizontal shaft discharge spiral case type, and is rated to develop 700 H.P. under 63 ft. net head, speed 600 R.P.M. Directly connected at each end to a centrifugal pump, the turbine unit has a discharge elbow coming off either side. This turbine, like larger Leffel turbines throughout the world, harnesses water power efficiently and economically.

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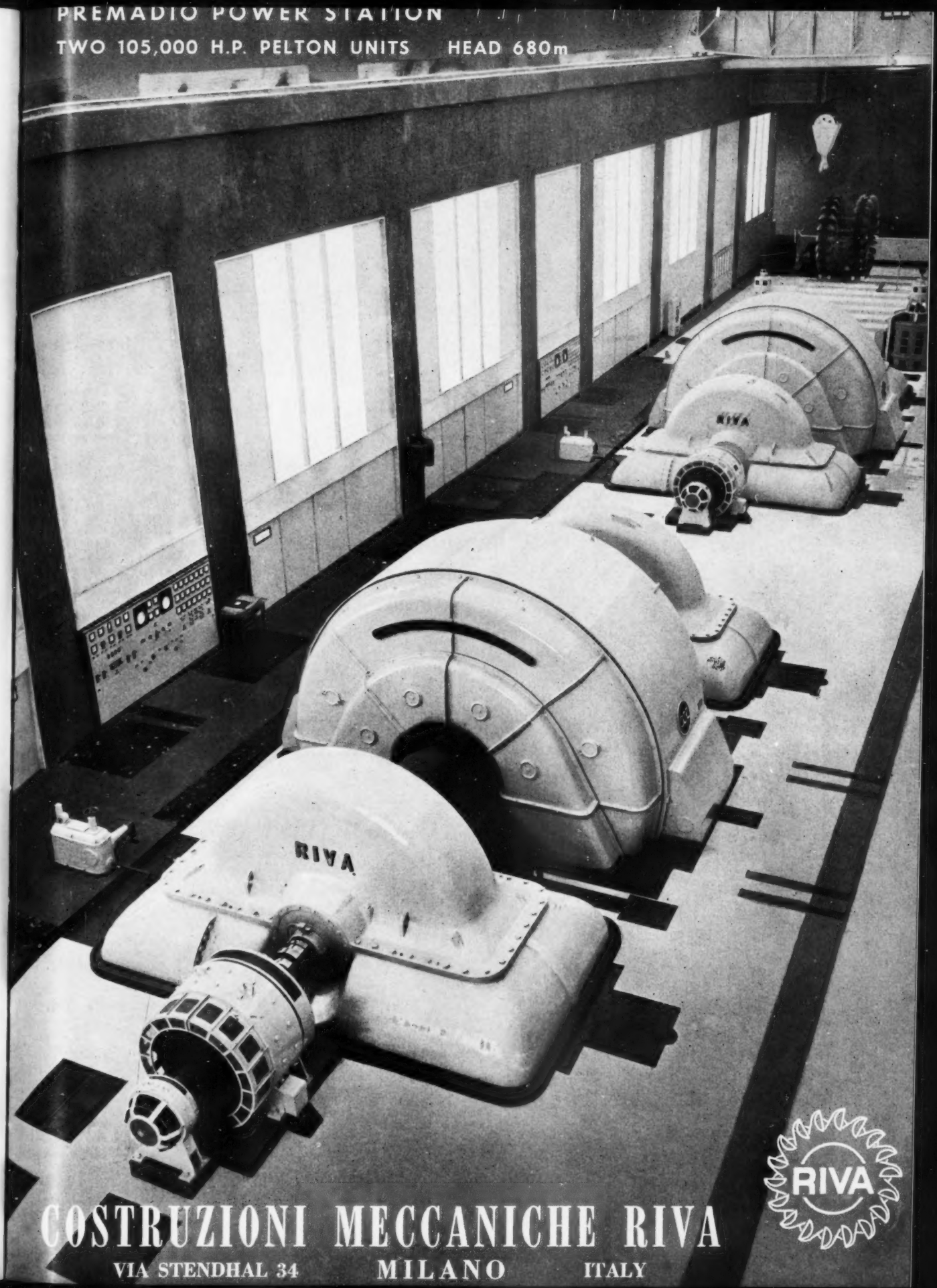
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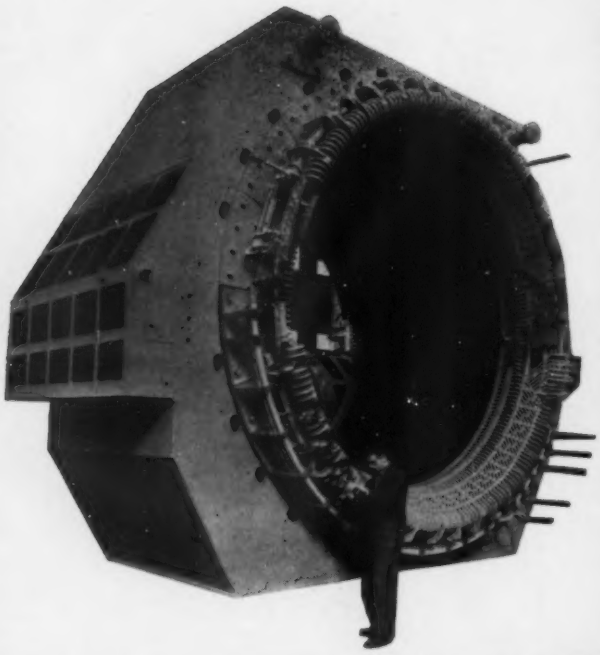

The stator of the 60MW hydro-alternator at Aura Power Station, Norway.

Wealth

G.E.C.

power

Wherever there's water in abundance, there's power potential in abundance too. In Scotland, the G.E.C. is helping to harness this power and has supplied the North of Scotland Hydro-electric Board with three 25MW hydro-alternators and control gear for the automatically controlled Errochty Power Station—the largest in the Tummel-Garry scheme.

Norway is another of the water-lands. Here, deep in the mountainside at the head of the Sunndalstjord is the Aura Power Station. And here a 60MW hydro-alternator has been installed for the Norwegian State Power Company by the G.E.C. In Tasmania, too, the G.E.C. has recently installed five 25MW hydro-alternators at Tungatinah.

To Scotland . . . to Norway . . . to Tasmania . . . to the water-lands of the world, the G.E.C. sends hydro-electric equipment for generation, control and distribution of power.

n of the water-lands

Three 25MW vertical shaft alternators at Errochty Power Station, Scotland

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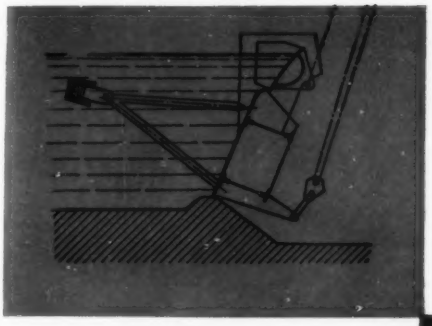

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9 Flap-topped segment gates of 16 m clear width and 8.00 m water head, installed in 3 weirs across the River Lech at Rain near Augsburg.
Illustration shows the gate with superposed regulating flap, seen from downstream. This type of gate, suspended by means of traction rods, from the pier-head in pivoted arrangement, is employed for major water heads. Drive by means of hoists or oil pressure. This gate allows of the adoption of slender, low piers without grooves and of statically favourable load conditions.

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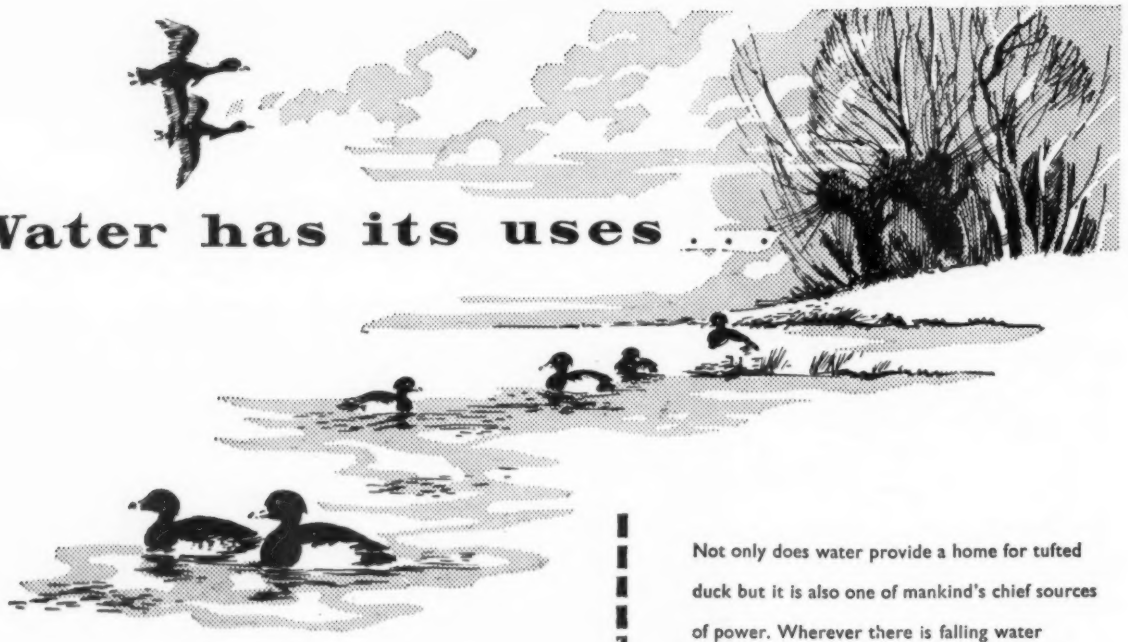

TUFTED DUCK (Nyroca Fuligula)
Found on fresh water throughout Great Britain; floats idly all day and flies to its feeding grounds at night. Food: water plants, insects and small fishes. Nests in colonies in rushes or coarse grass and raises eight to twelve young each season.

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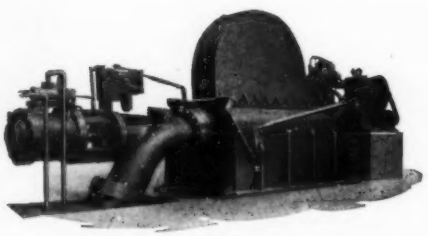

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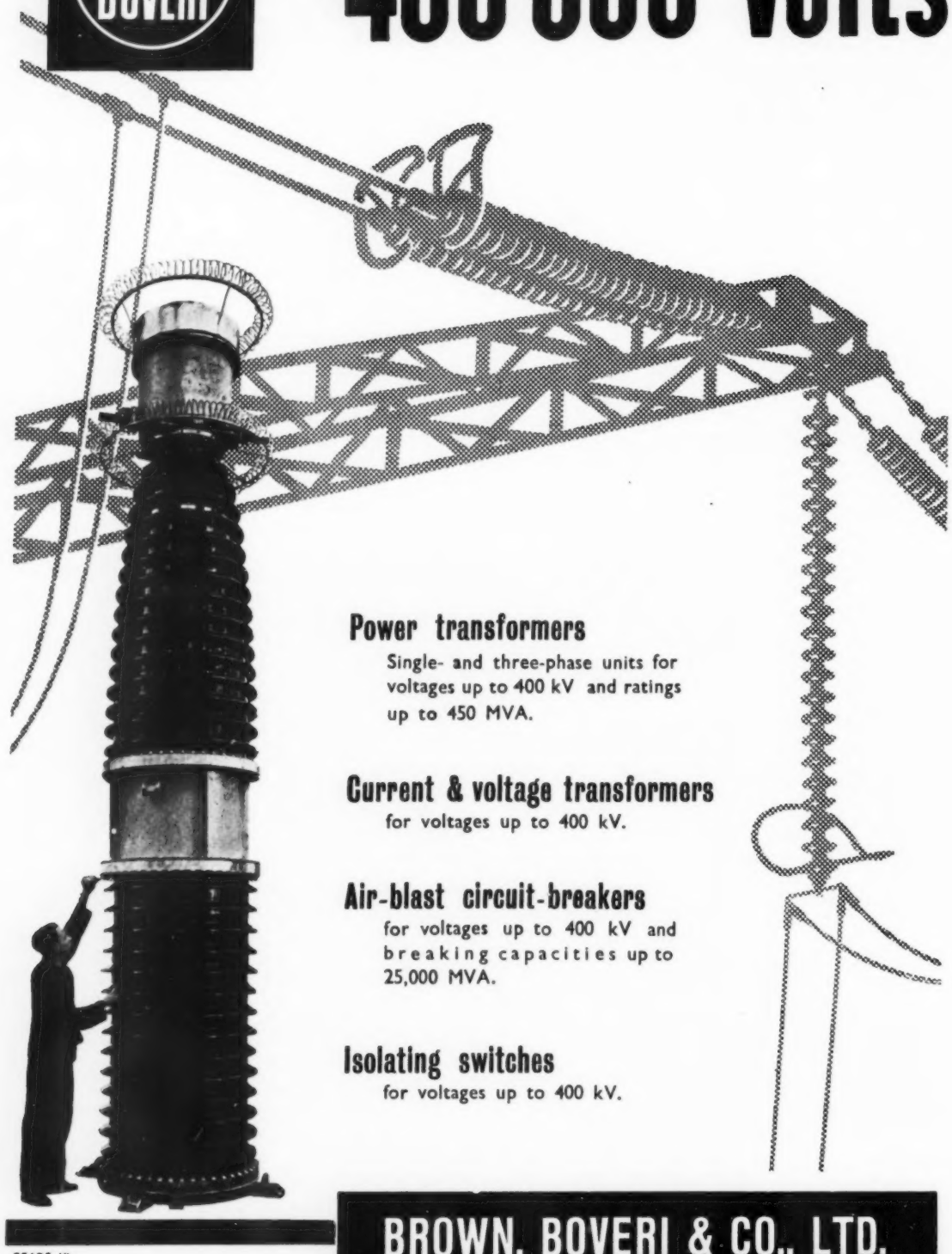

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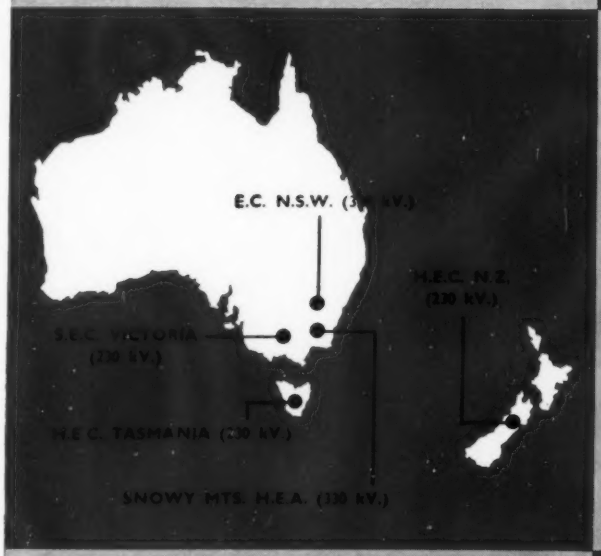

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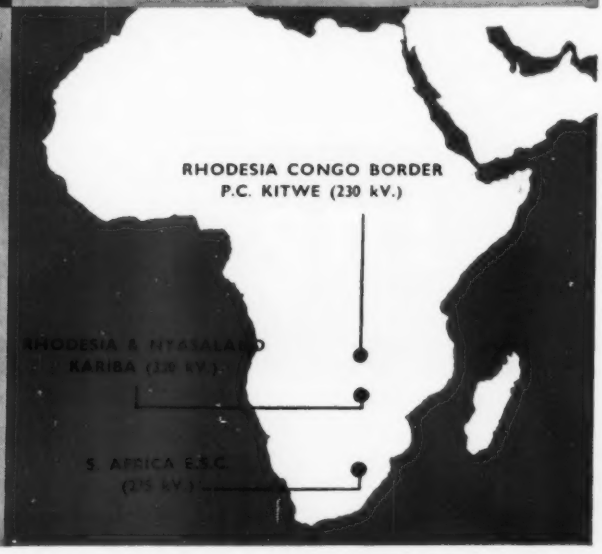

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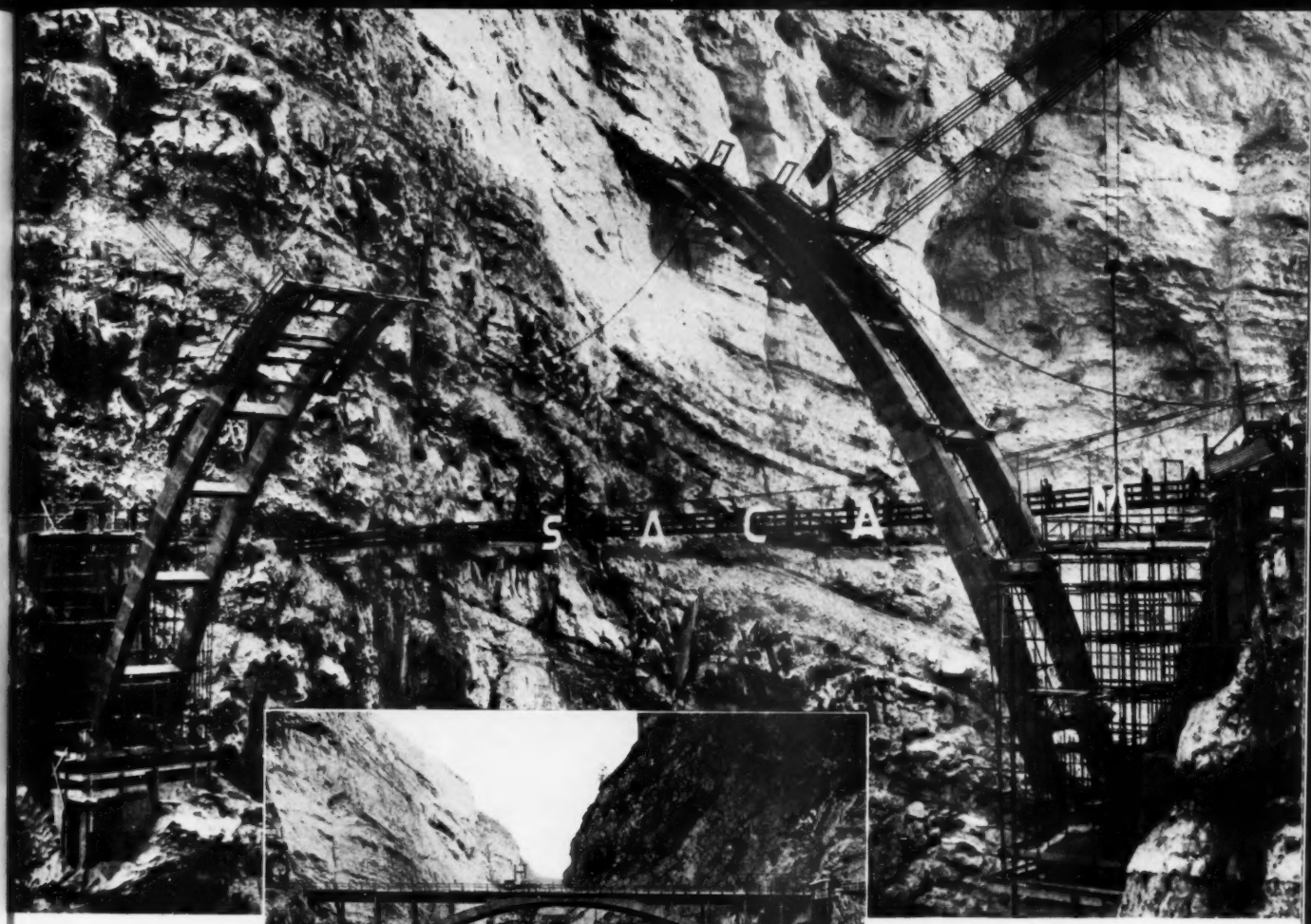

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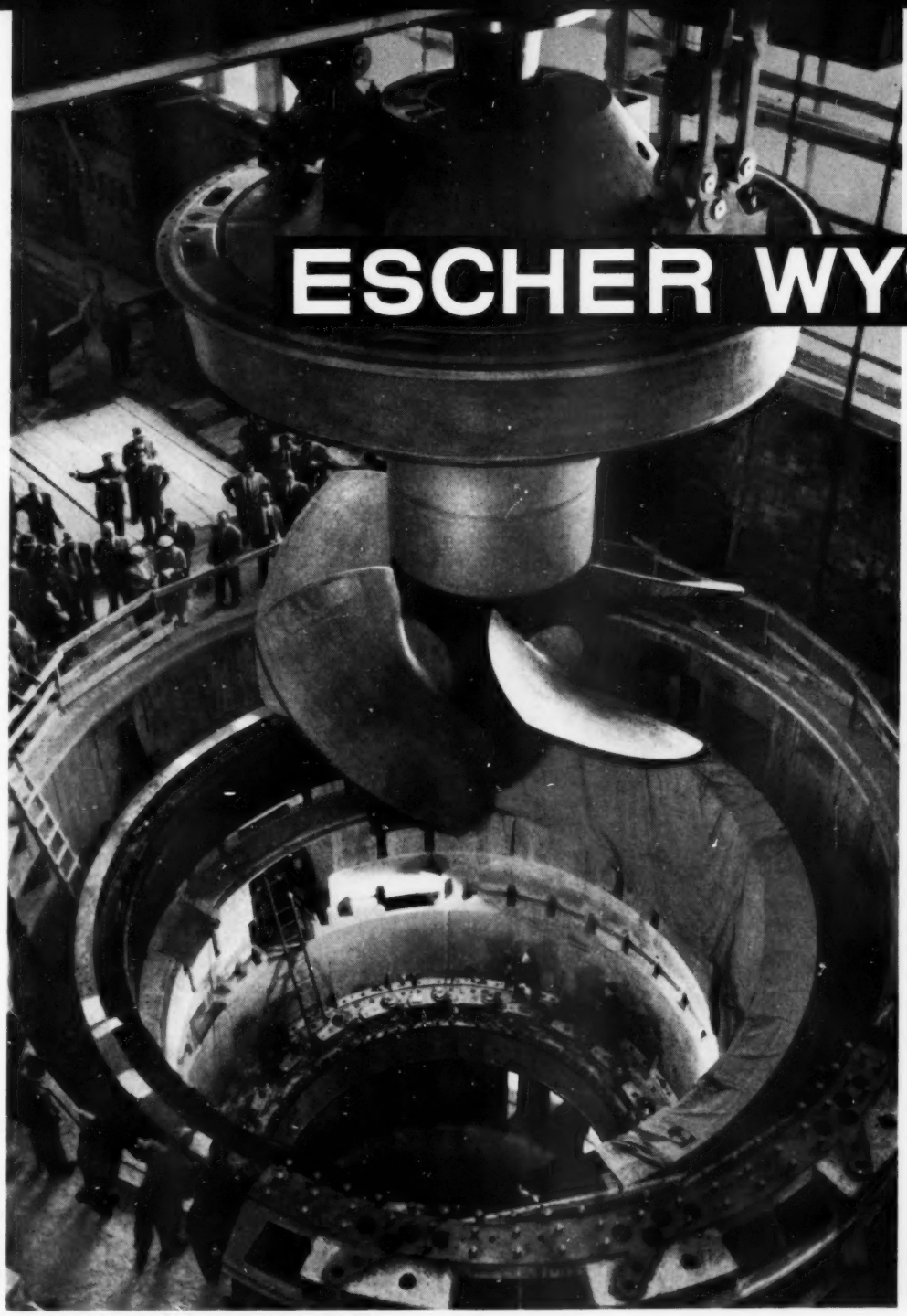

Lowering the runner wheel into one of the Escher Wyss Kaplan turbines at the Rheinau Power Station (Switzerland)

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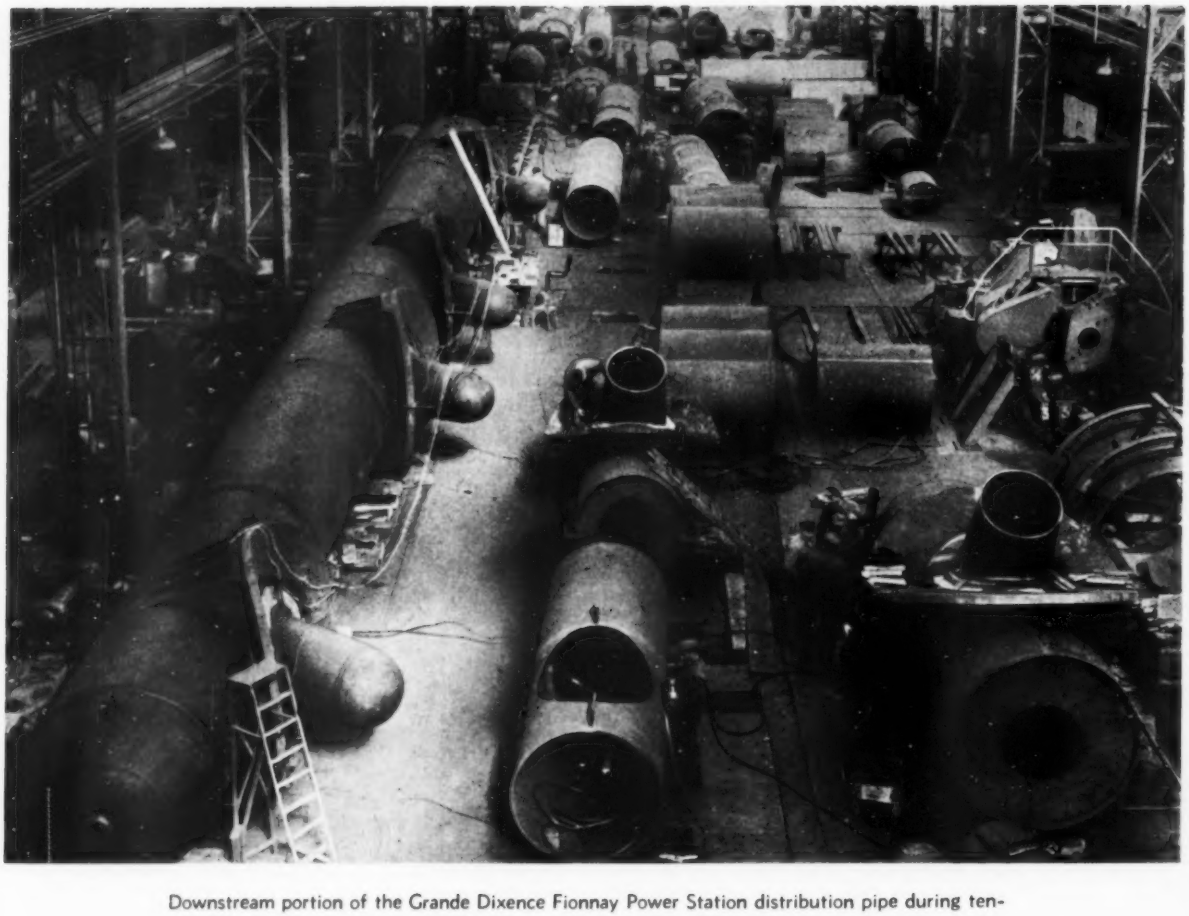

Downstream portion of the Grande Dixence Fionnay Power Station distribution pipe during tensometric shop tests. Max. wall thickness 52 mm., service pressure 96 at., test pressure 121 at.

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Speed: 150 r. p. m Runner diameter: 12 feet 6 ins.
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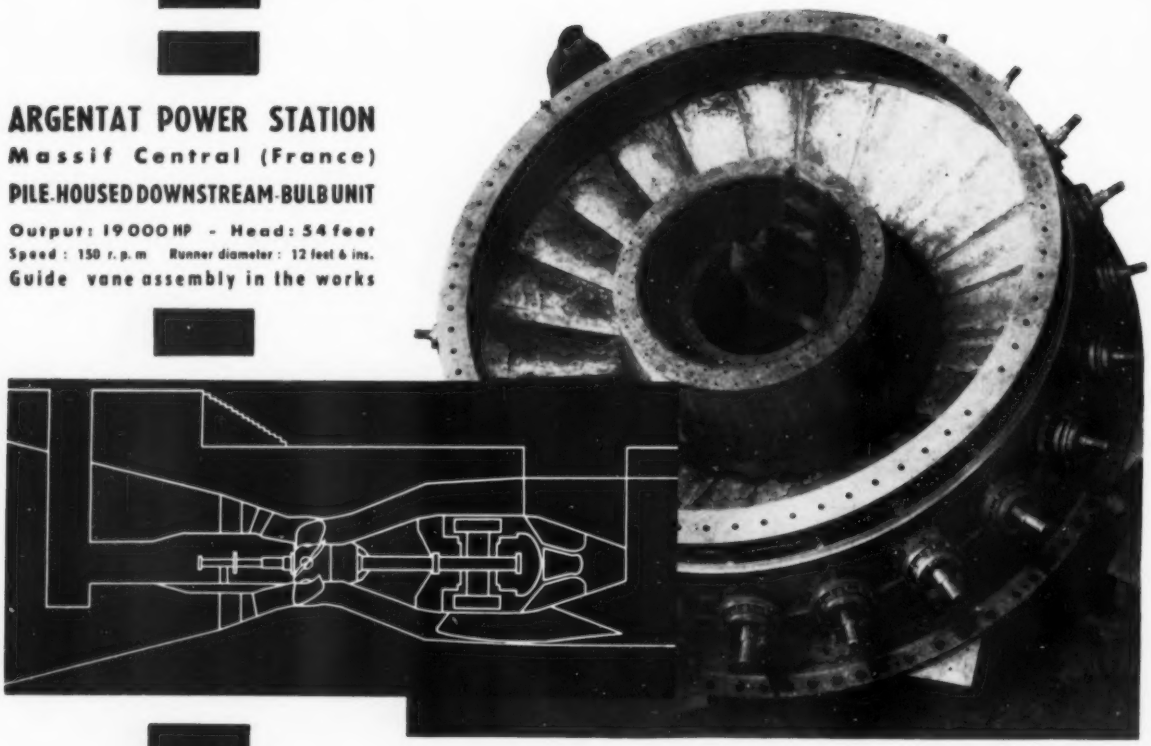

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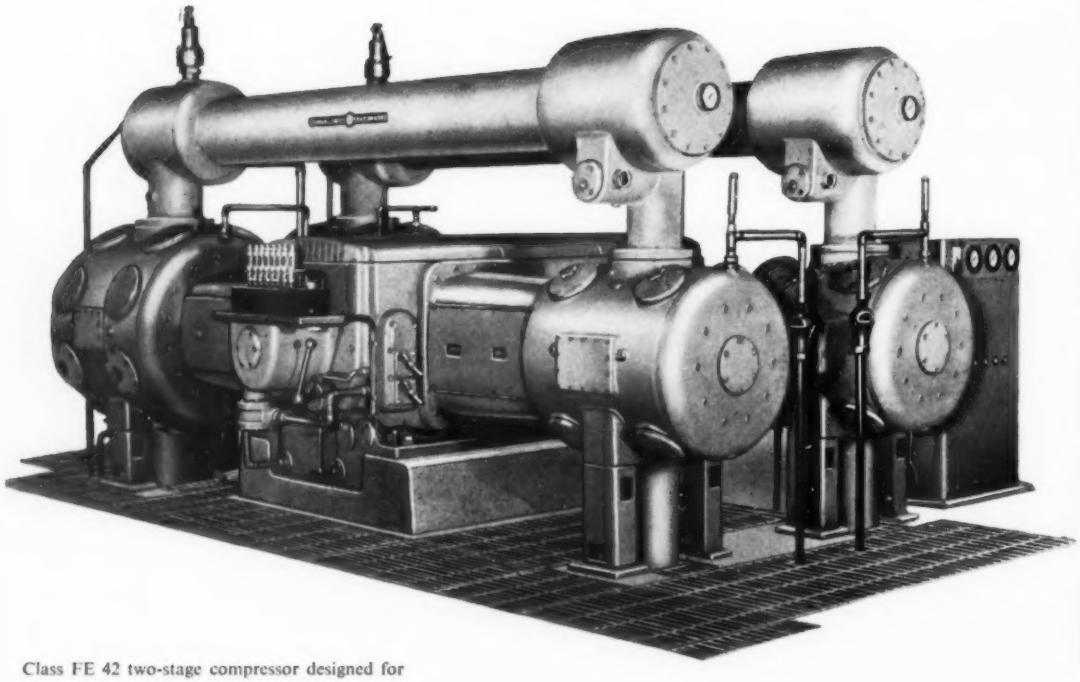

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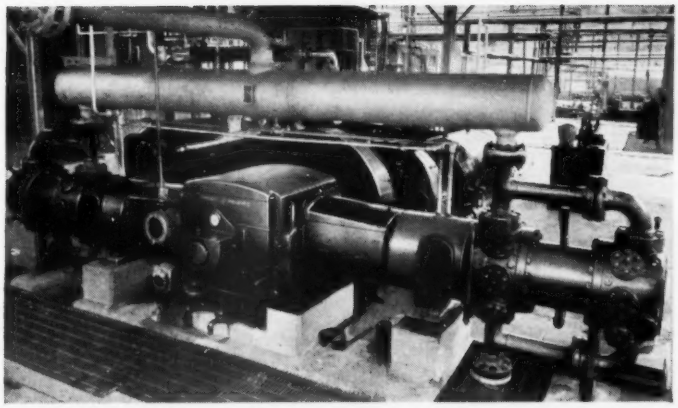
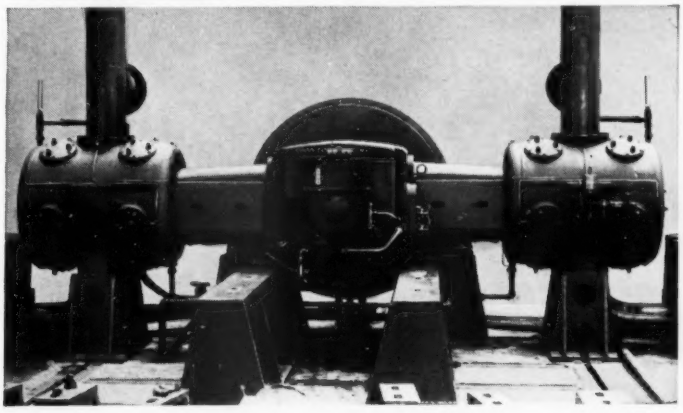


Class FE single-stage non-lubricated hydrogen recycle compressor boosting to 660 p.s.i. at work on refinery service.

A Class FE compressor for low pressure operation on the test bed in the CP works at Fraserburgh, Scotland.



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Convinced of the advantages of Dresser Couplings, the B. C. Electric Company, since 1948, has installed four 100 per cent Dresser-Coupled penstocks at Bridge River, British Columbia. On all four projects, the use of Dresser Couplings resulted in faster completion of the line and assured permanently tight, maintenance-free joints. This meant earlier revenue from plant operation . . . uninterrupted service to power consumers.

Throughout the world, leading engineers and contractors recognize the superiority of the Dresser pipe joining method — permanent tightness, flexibility, simplicity, strength and economy.

Even highest pressure penstocks are faster and easier to install when Dresser Couplings are used. The need for special skills and expensive equipment is minimized. In many cases, a field crew of a few men can cut construction time from months to weeks through the use of Dresser Couplings. Work proceeds in any weather with wrenches the only tools needed.

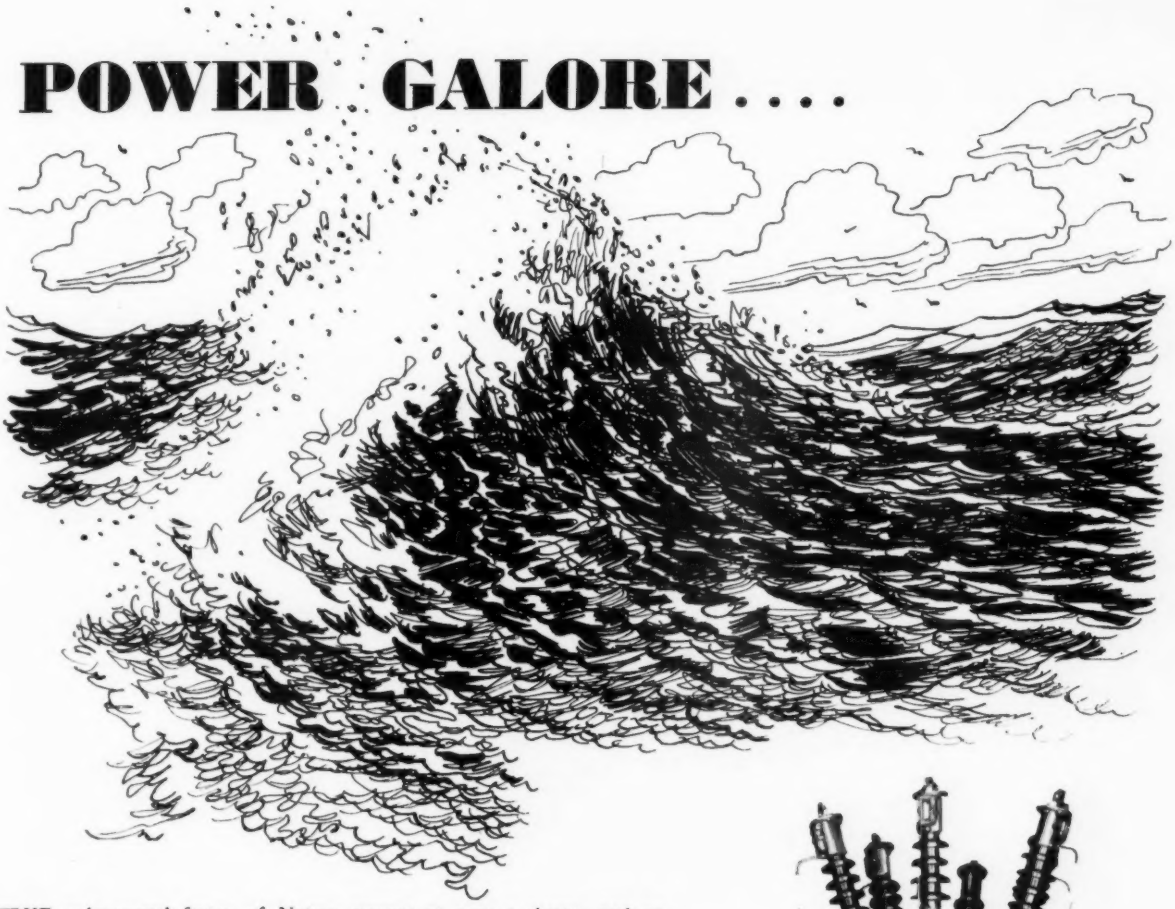
Where required, high tensile steel permits thinner pipe walls and lower pipe weights. This means easier handling. In some cases, up to 25 per cent in weight is saved.

But a Dresser-Coupled penstock's most important advantage is permanence . . . it stays bottle-tight for life. The resilient gaskets built into every coupling assure positive sealing, eliminate expansion-contraction problems, permit joint deflections so the penstock may follow the natural contour of the right of way.

For your next penstock project, send for helpful, interesting engineering data on the successful and varied Dresser-Coupled steel penstock installations. Write Dresser Manufacturing Division, Bradford, P.a., U.S.A.

ONE OF THE DRESSER INDUSTRIES

POWER GALORE . . . .



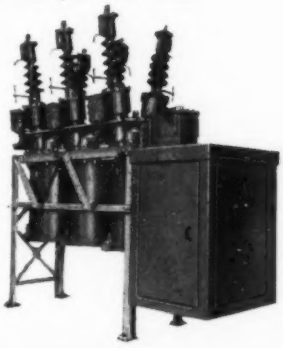
THE unharnessed forces of Nature present at once a threat—and an opportunity! We are concerned only with the latter, for it is the safe control of electric power, at all stages, that enables full use to be made of this natural force.

In this connection, "safe control," is the function of "Ferguson Pailin" Switchgear; this is installed at strategic points in supply system which in so many parts of the World, rely on water power as a motive force.

For example, switch units similar to the type ROP32 oil circuit-breaker shown, designed for 22 kV or 33 kV service and with an A.S.T.A. certified breaking capacity of up to 500 MVA, are operating satisfactorily in many outdoor substations, sometimes under extreme climatic conditions.

Particular attention has been given in the design of our outdoor circuit-breakers to the provision of maximum protection of vital parts against humidity, heat or cold.

Especially would we draw attention to the adaptability of this Switchgear to particular system requirements—a feature which is a byword among those familiar with "Ferguson Pailin" products.



Another popular type of outdoor switch unit is the TTPO24 oil circuit-breaker, a triple-tank, frame mounted unit, with an A.S.T.A. certified performance of 1500 MVA at 33kV

Type ROP32 Oil Circuit-breaker

Ferguson Pailin LIMITED *for Switchgear*

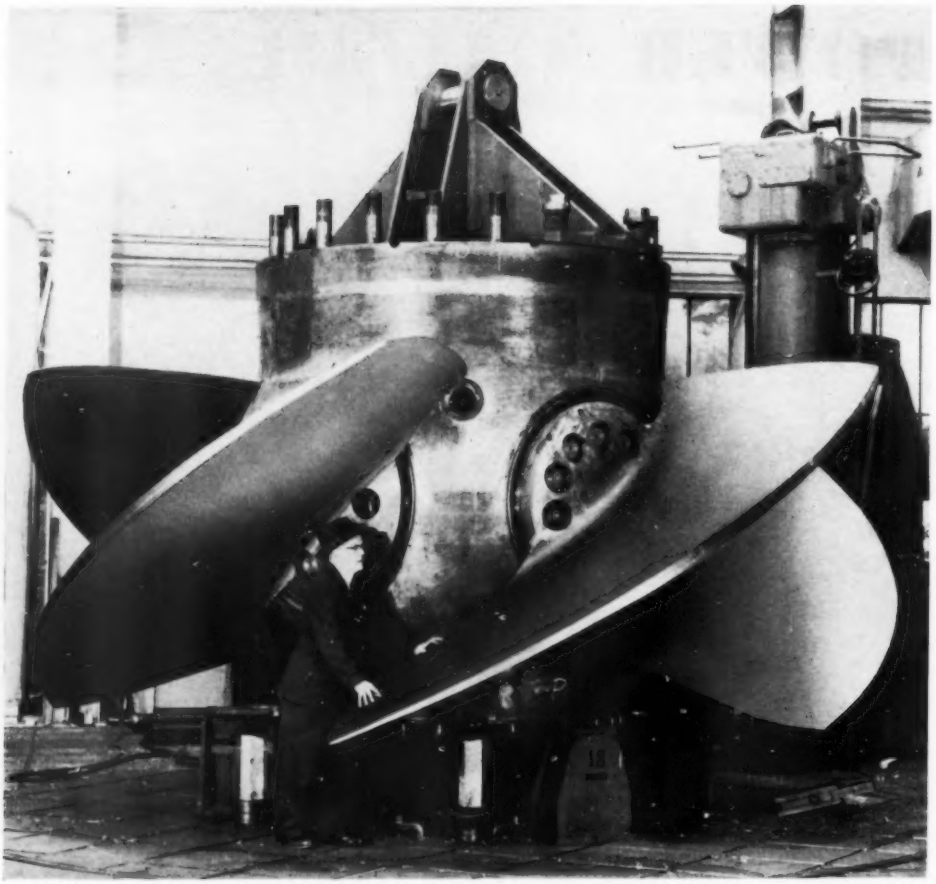
An A.E.I. Company

Head Office & Works: **HR. OPENSHAW MANCHESTER 11,** *Telephone :* DROylsden 1301 (Pte.Branch Ex)

LONDON OFFICE : Bush House, Aldwych, W.C.2 BIRMINGHAM OFFICE : Windsor House, 656 Chester Road, Erdington, 23 GLASGOW OFFICE : Central Chambers, 109 Hope Street, C.2.

REPRESENTED IN PRINCIPAL OVERSEAS TERRITORIES

Runner for The Panchet Hill Power Station. Runner diameter 5·4 m. (17 ft. 8 in.)



NOHAB turbines

. . . always a step ahead!

Specifications of large water turbines already delivered, or under delivery by NOHAB

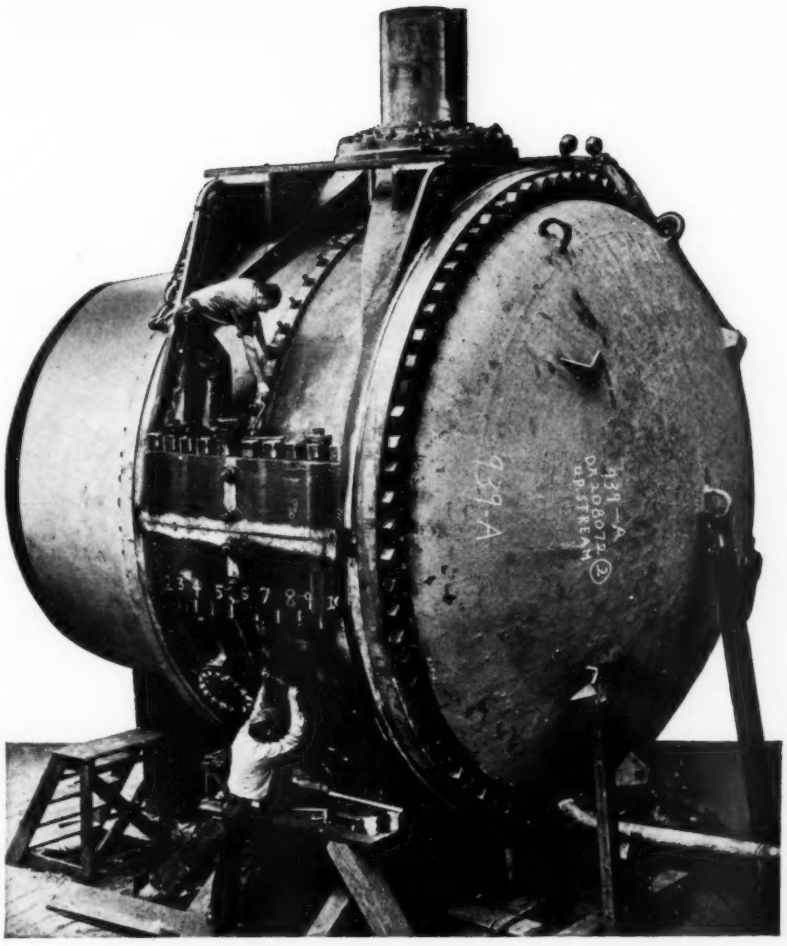
Power station	Number and type of turbine	Head ft.	Output per turbine, h.p.	Speed r.p.m.	
Stornorrfors, Sweden In service 1959	3 Francis	239·5	178,120	125	**Guaranteed output at 246 ft., 200,000 h.p. The most powerful turbines in the world.**
Lasele, Sweden In service 1956	2 Kaplan	170·6	87,200	150	**This is the highest head for which a Kaplan has been installed in Sweden.**
Porsi, Sweden In service 1960	2 Kaplan	118·0	131,000	115	**These will be the most powerful Kaplans in Sweden.**
Mael, Norway In service 1957	1 Francis	140·4	47,500	150	**With regard to dimensions, the largest turbine in Norway.**
Panchet Hill, India In service 1958	1 Kaplan	81·0	56,000	125	**India's largest Kaplan, in output as well as dimensions.**
Rincón de Baygorria, Uruguay In service 1959	3 Kaplan	48·2	49,500	79	**With regard to dimensions, South America's largest turbines.**

NOHAB

NYDQVIST & HOLM AKTIEBOLAG

TROLLHÄTTAN SWEDEN

Cables: NOHAB Telex: 5284 Telephone: 18000



This 16-Foot Butterfly Valve Illustrates the type of work which Newport News takes in stride. Newport News built 3 such valves, each weighing 446,000 lbs., for the Ross Power Plant, Skagit Project, Department of Light, City of Seattle, Washington. Designed for a water flow of 3,620 cu. ft. per sec., and a hydrostatic pressure of 290 psi., these valves were shop tested by Newport News at 450 psi. They are hydraulically operated with oil at 1,500 psi. pressure. Shop tests assure speedy, trouble-free assembly of Newport News built equipment, on the site.

The TEST of a TITAN

Here is one of the largest high head butterfly valves ever built, undergoing a shop test at Newport News. If you had an opportunity to follow this unit from start to finish, you would see *first hand* how Newport News produces massive equipment *economically*. For economy is a basic advantage that results from Newport News' high integration of skill and production facilities.

Large engineering and technical staffs, operating a plant comprising acres of brass, iron and steel foundries, five huge machine shops and other extensive fabricating facilities, have made Newport News one of the world's largest producers of hydraulic turbines, valves, gates, penstocks and other essential equipment... both standard and special in design.

Let us bid on your equipment. Write us today for your copy of ***"Water Power Equipment."***

NEWPORT NEWS

SHIPBUILDING AND DRY DOCK COMPANY
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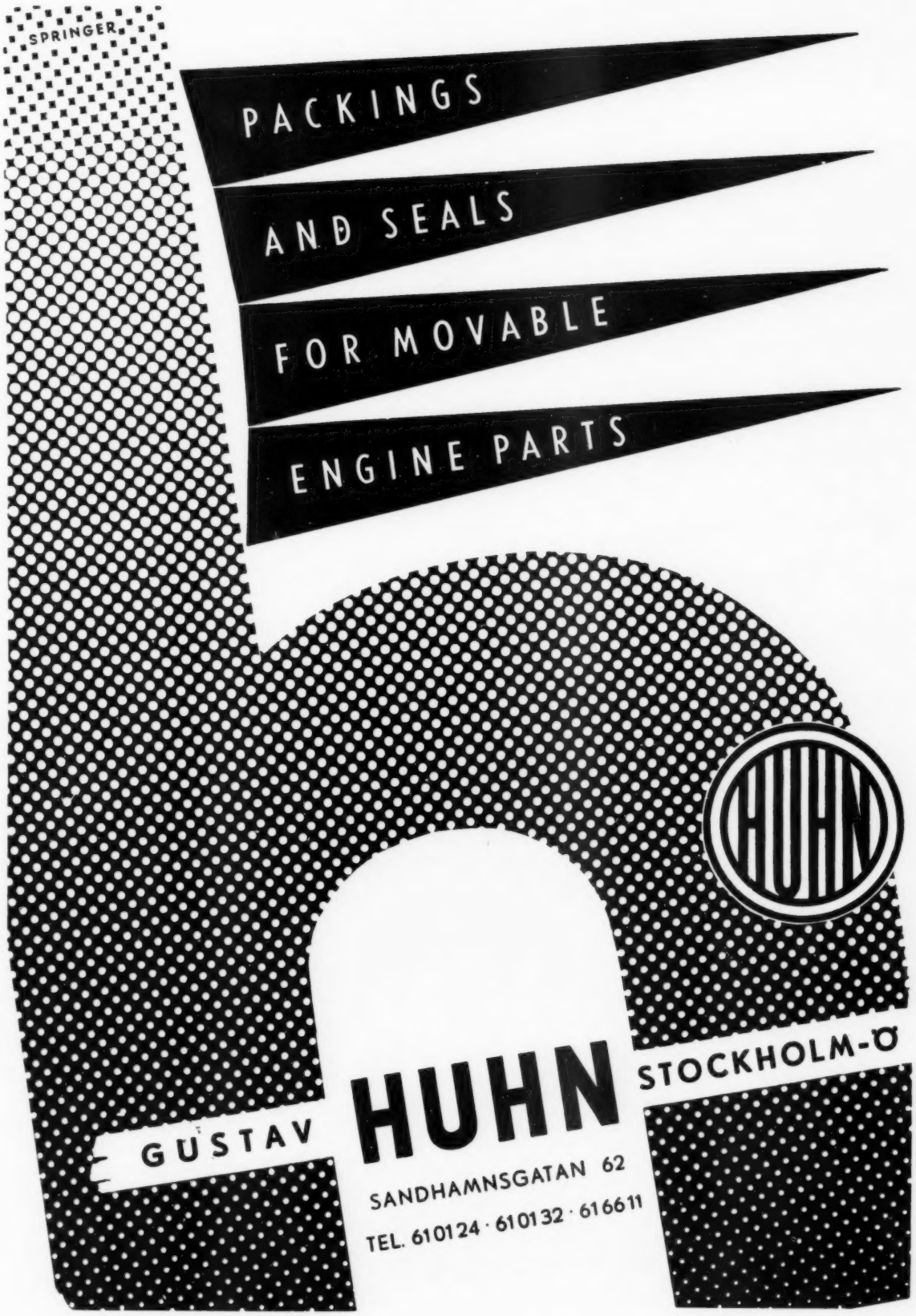
BRECO 7½ ton, Standard Single Motor Radial Travelling Cableway, installed in Colombia for the construction of the Anchicaya Dam.

BRECO Ropeways and Cableways have been supplied and installed for operation under most extreme climatic and topographical conditions throughout the world.

BRECO—Sole Manufacturers of 'Travelift' Cableways, outside the North American Continent. 'Travelifts' are especially suited to high speed heavy construction work.

Literature available on request.

BRITISH ROPEWAY ENGINEERING CO. LTD. Plantation House, Mincing Lane, E.C.3. Tel: Mincing Lane 7901. Tel. Address: Boxhauling, Fen, London.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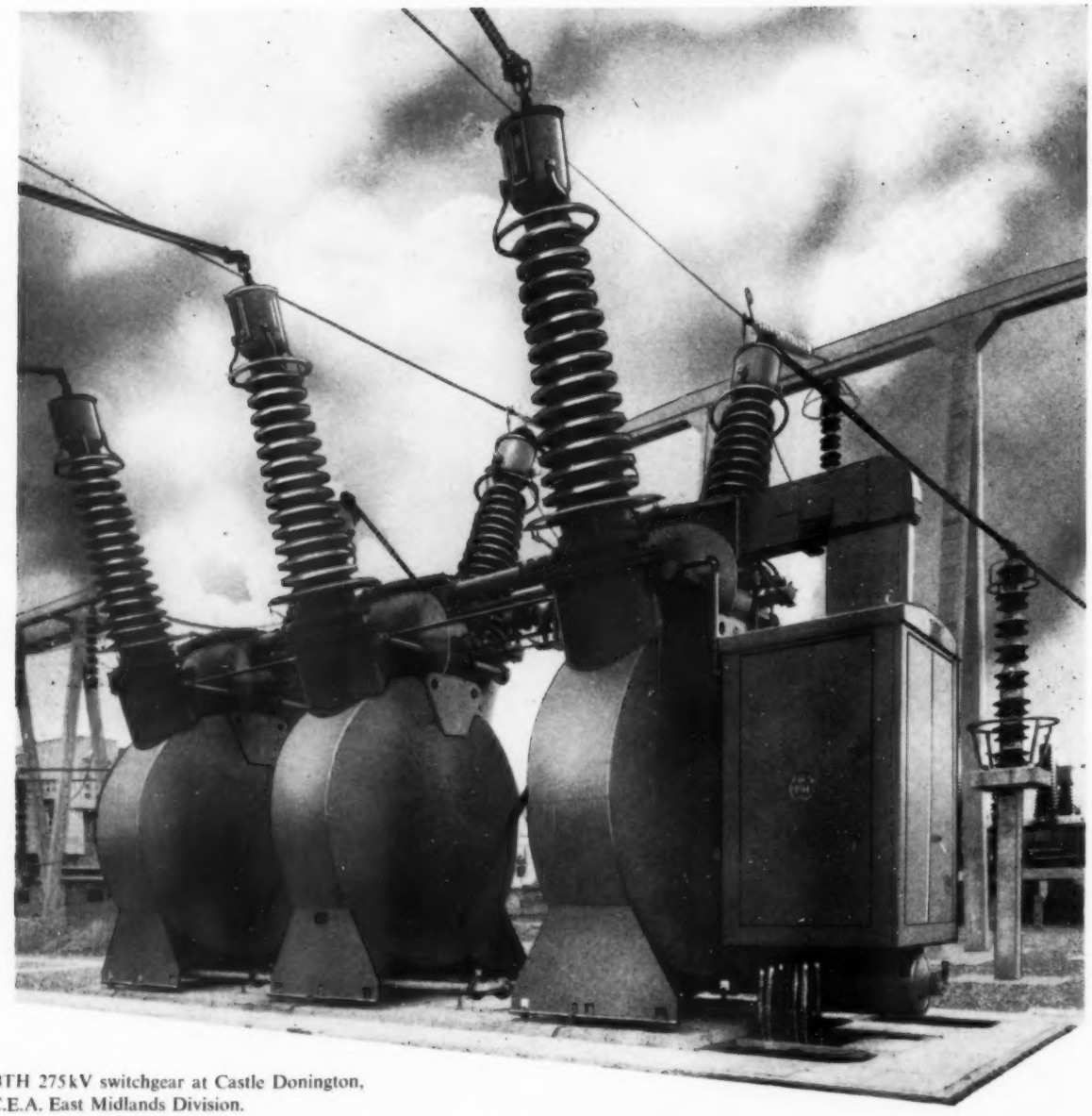

BTH 275kV switchgear at Castle Donington,
C.E.A. East Midlands Division.

B
T-H

330-kV SWITCHGEAR
for
KARIBA

The two million pound order awarded to British Thomson-Houston for the Kariba Hydro-electric Scheme includes 23 oil circuit-breakers of the BTH lenticular type. These units are a direct development of those installed on the 275kV British Super-Grid.

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THE BRITISH THOMSON-HOUSTON COMPANY LIMITED · WILLESDEN · ENGLAND

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on crawler-carriage,
bucket capacity 10 cu. ft.

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HL 280/R

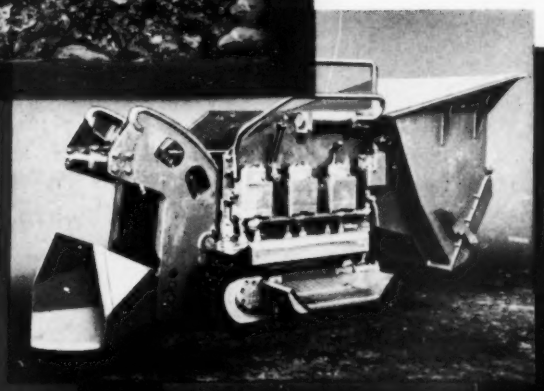
Manoeuvrable to the very spot you want to clean up.

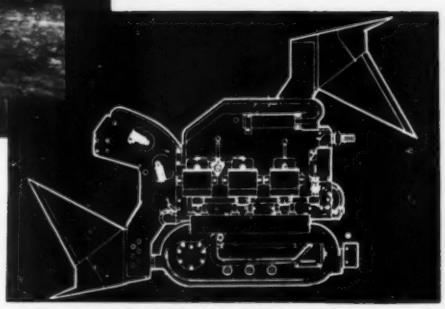
The bucket being wider than the carriage, it can be forced into the heap without being blocked by crawlers.

Ample clearance below bottom of carriage.

Bucket automatically fixed in lifted position during transport.

Discharge height up to 5 ft.





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PENSTOCKS
WATER GATES
HYDROELECTRIC EQUIPMENT

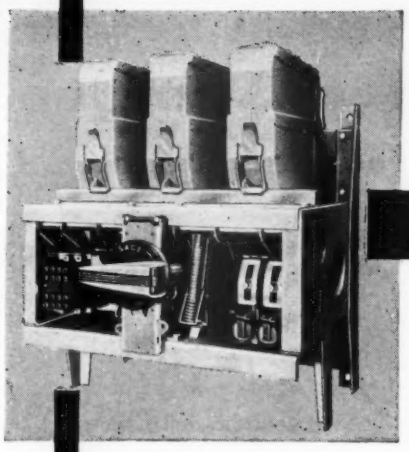
CAP DE LONG: Self-hooped penstock for the Pragnères Power Station, in the Pyrénées.
Head 1,252 metres, Flow 14,6 cu.m./sec.
Diameter 2.25 to 1.60 metres, Power 200,000 H.P.

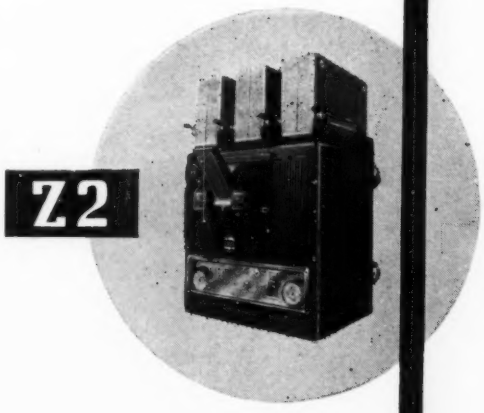
PASSY: Penstock bridge over the River Arve, in the French Alps.
Span 72 metres, Diameter 3 metres.

ETABTS. BOUCHAYER & VIALLET
SOCIETE DAUPHINOISE d'ÉTUDES & de MONTAGES
GRENOBLE - FRANCE

UPT - SACE 120

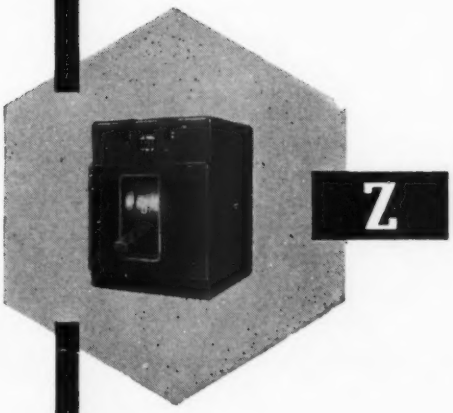
Circuit-Breaker type "P"
Rated voltage 600 V
Rated current up to 4,000 A
Breaking capacity up to 55 kA
at 500 V a.c.





Circuit-Breaker type "Z 2"
Rated voltage 600 V
Rated current up to 600 A
Breaking capacity up to 20 kA
at 500 V a.c.

Circuit-Breaker type "Z"
Rated voltage 500 V
Rated current up to 150 A
Breaking capacity up to 12 kA
at 500 $\frac{1}{2}$V a.c.



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In all parts of the world Bucyrus-Erie excavators are preferred for their output capacity and stamina. Three 54-B diesel shovels with Torque Converter operate in this limestone quarry at Ain Sebaa on the outskirts of Casablanca. They maintain consistently high outputs loading rock for the manufacture of cement. There is a size of Bucyrus-Erie heavy-duty excavator for your job also.

BUCYRUS-ERIE COMPANY
South Milwaukee, Wisconsin, U.S.A.

RUSTON-BUCYRUS LIMITED, LINCOLN, ENGLAND

BUCYRUS ERIE

More than 20 million KMW horsepower in 50 countries

90,700 h.p. runner for one of the four KMW Francis turbines for San Esteban.

The Spanish power station of San Esteban belonging to the Saltos del Sil S.A., where the fourth KMW unit has recently been put into service.

KMW started manufacture of water turbines in 1880 and had up to the beginning of 1958 received orders for 2,765 units with a total output of 12·4 million horsepower, of which 4·7 million horsepower are Kaplan turbines. If the KMW turbines built by our licensees are included, these totals can be increased to 20 and 7 million horsepower respectively. These figures show that KMW turbines are in great demand for power stations all over the world.

Among the turbines recently delivered or ordered can be mentioned:

Number of units	Type	Client	Plant	Country	Total output h.p.	Net head feet	r.p.m.
2	Kaplan	Bålforsens Kraft AB	Bålforsen	Sweden	102,220	102	150
2	Pelton	Hjartdöla Kraftverk	Hjartdöla	Norway	142,000	1,820	428
1	Francis	A. Johnson & Co. (Canada) Ltd.	Willson	Canada	82,000	263	180
2	Kaplan	Swedish State Power Board	Järkvissle	Sweden	118,920	45	75
3	Kaplan	Bjurfors AB	Bjurfors	Sweden	101,550	64	125
3	Francis	Empresa Nacional de Electricidad S.A.	Pullinque	Chile	67,200	156	250
2	Francis	Stora Kopparbergs Bergslags AB	Trängslet	Sweden	280,000	459	214
2	Kaplan	Graningeverkens AB	Kvistforsen	Sweden	183,000	164	150
2	Francis	Stockholms Superfosfatfabriks AB	Parteboda	Sweden	42,700	105	167
2	Francis	AB Mölnbacka—Trysil—Höljes Kraftverk	Höljes	Sweden	182,800	284	187·5

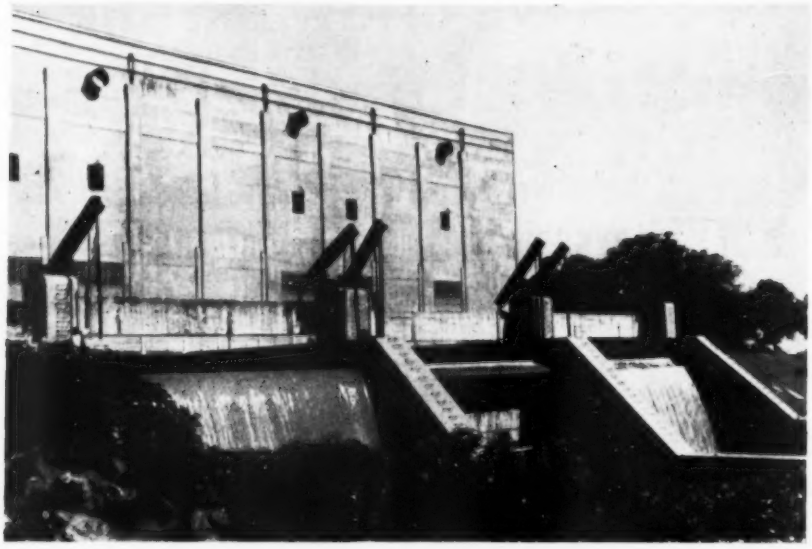
AB KARLSTADS MEKANISKA VERKSTAD

KARLSTAD — SWEDEN

Representatives in Canada: A. Johnson & Co. (Canada) Ltd., 607 Shell Tower Building, Montreal 2.

KMW

At the Power House



TAILRACE GATES

When it is necessary to create an artificial tailrace pool and maintain it at a constant level, the requirements can often be best met by the use of some form of automatic gate; the design and manufacture of which can be readily undertaken by Glenfields, who have a wealth of experience in this type of work.

The site illustration shows 42 ft. span x 7 ft. deep automatic Tilting Gates on the outlet of the tailrace pool of Mohammedpur Power Station in India. The construction of such gates is illustrated by the shop photograph.

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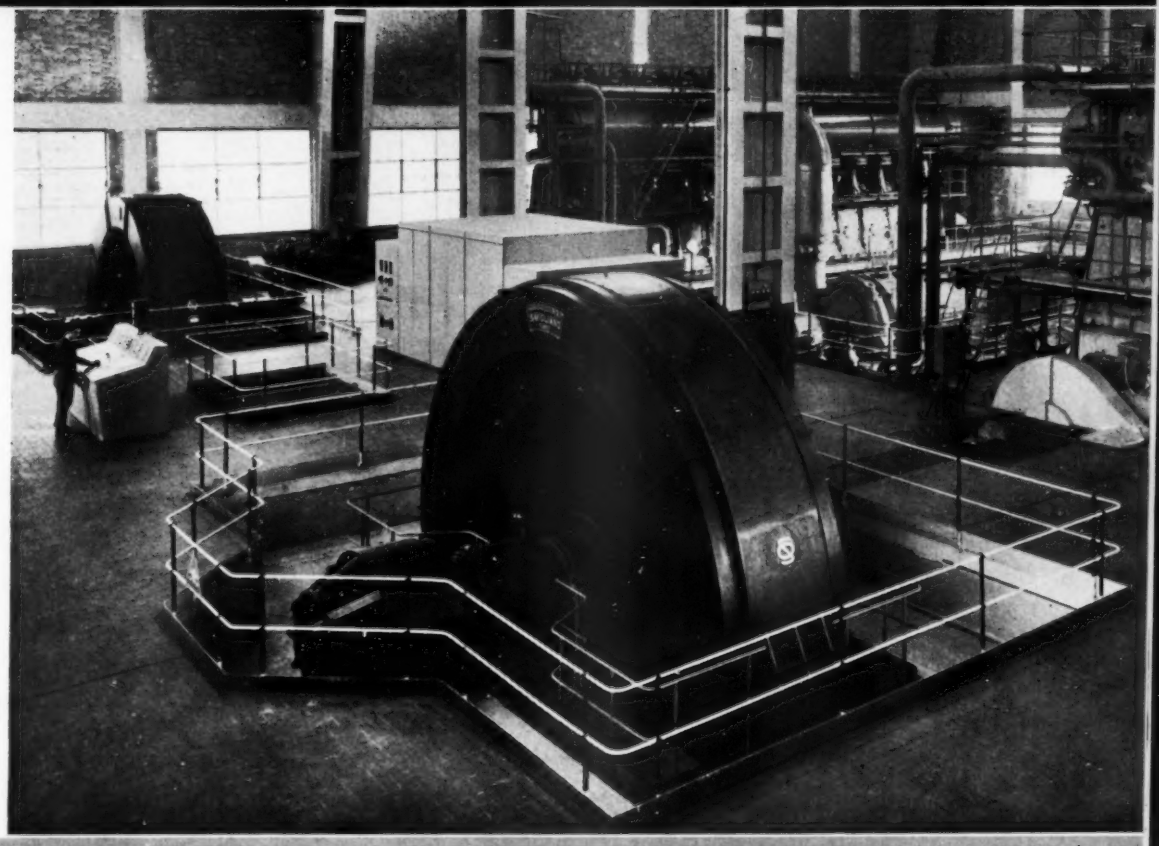

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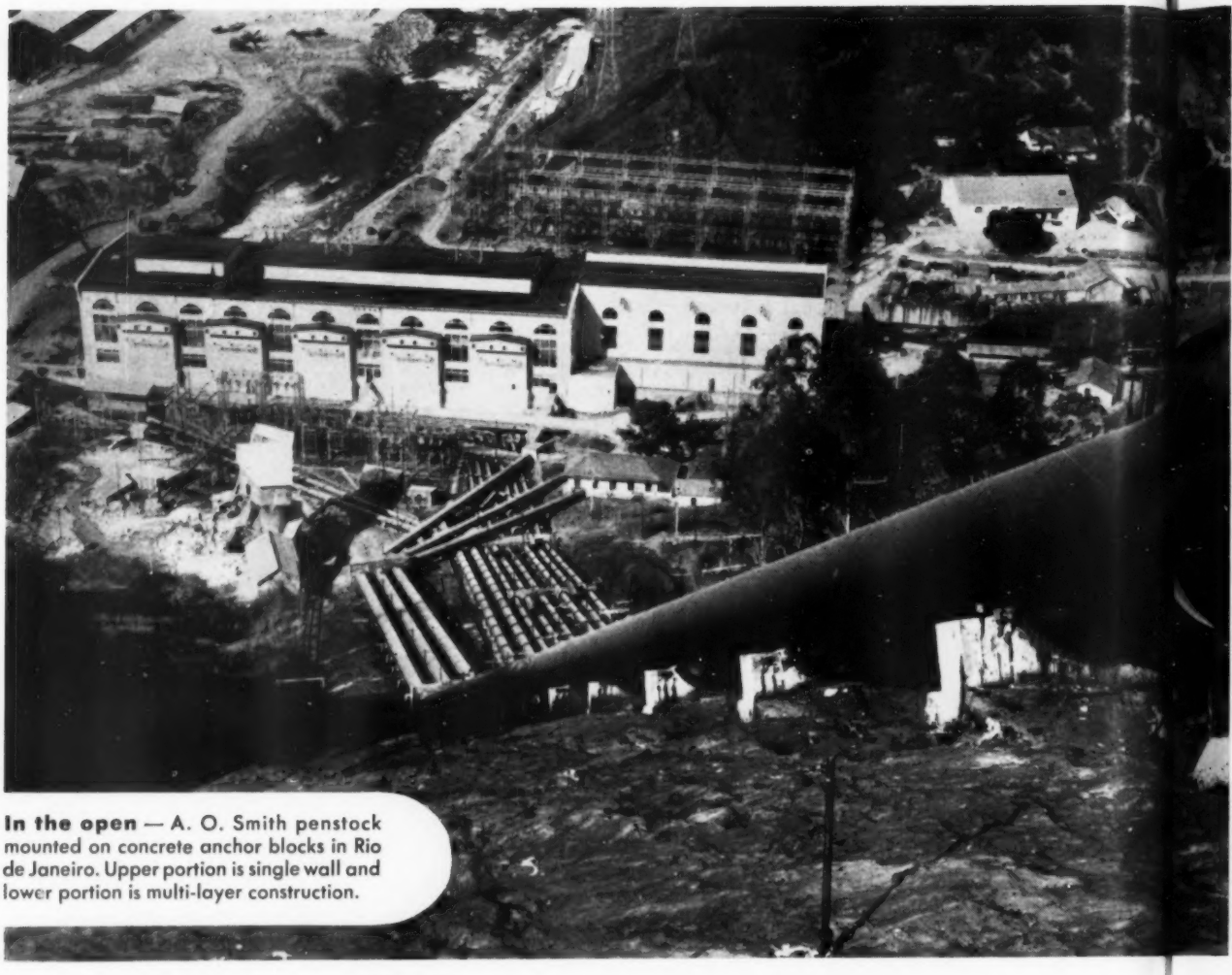
corso mortara 4
torino



In the open — A. O. Smith penstock mounted on concrete anchor blocks in Rio de Janeiro. Upper portion is single wall and lower portion is multi-layer construction.

In the open . . .

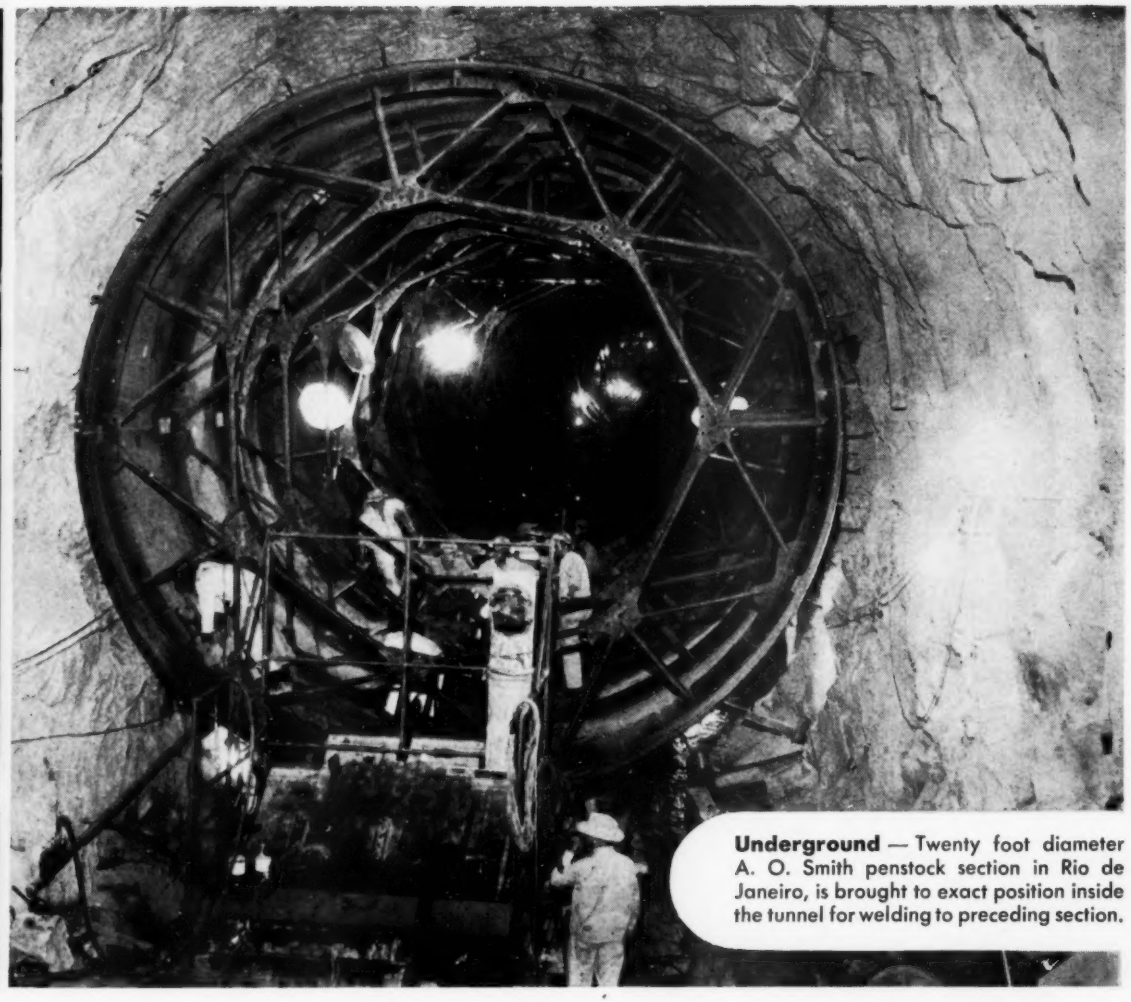
A. O. Smith engineered penstocks . . . backed by rigid customer protection controls . . . meet the challenges of high hydrostatic heads, large diameters

The proven performance record of A. O. Smith welded steel penstocks results from this ideal combination: Rigid customer protection controls and depth of experience gained as one of the world's largest manufacturers of welded steel pipe and pressure vessels.

Especially engineered for medium and high heads . . . small and large diameters . . . combinations of single wall and multi-layer construction, every penstock part is subjected to these quality controls. Components must conform to strict material specifications and welding procedures. Radiographic-examinations, stress relief, hydrostatic testing and all other quality checks are exceeded.

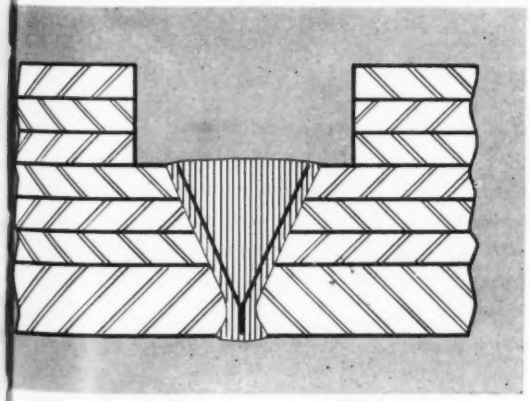
All penstocks for delivery outside the United States are completely shop-fitted and match-marked *before* shipment.

Put A. O. Smith penstock design experience to work for you! For further details, write A. O. Smith International S. A., Administrative Offices: P. O. Box 331, Milwaukee 1, Wis., U.S.A.



Underground — Twenty foot diameter A. O. Smith penstock section in Rio de Janeiro, is brought to exact position inside the tunnel for welding to preceding section.

or underground



Cross section of typical A. O. Smith MULTI-LAYER construction illustrating girth seam weld.

Exclusive* MULTI-LAYER *Construction

— saves up to 15% in the weight of steel required for high pressure penstocks of conventional design — with no sacrifice in safety factor. Multi-layer penstocks cost much less to ship and to install. Field welding is easier — requires no stress relieving.

Through research A.O. Smith *... a better way*

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POST INSULATORS for the Hirakud Dam project

S.P.P. Post Insulators were supplied for the 132kV. outdoor switchgear on the Hirakud Dam Project.

S.P.P. Post insulators have also been used on all the major hydro-electric projects in India, including the following:—

Damodar Valley Corporation
U.P. Government
Madras Government
Mysore Government
Travancore—
Cochin Government
Bombay Grid Scheme
Jog Falls Scheme
Machkund Scheme
North Calcutta Scheme
Pallivasal Scheme
Tungabhadra Scheme
Kunda, Madura, Mettur Dam, Moyar, Papanasam and many other substations on the Madras Grid.

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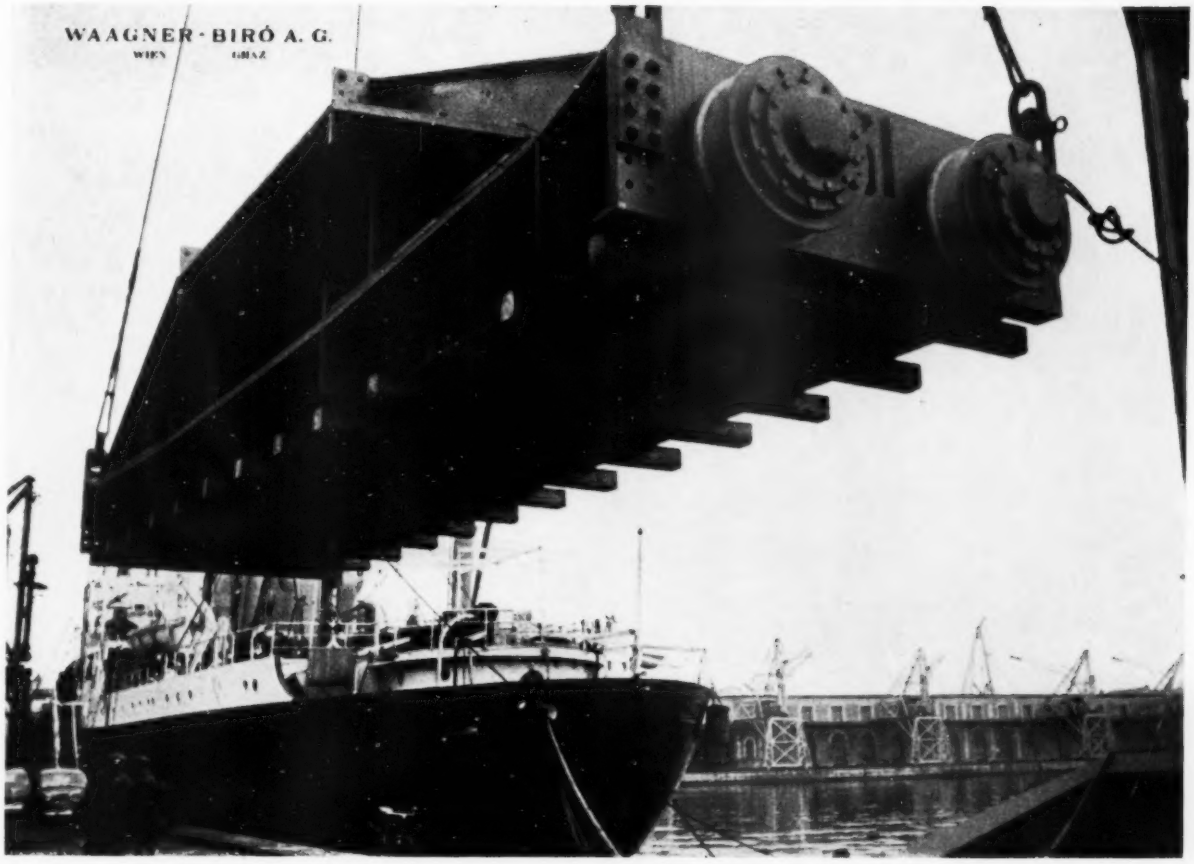

Transloading in the Port of Trieste of a part of an intake bulkhead gate for the Nile Power Station at Assuan. Five of these parts make up one complete bulkhead gate. Dimension of the gate 8 x 14 m.

●

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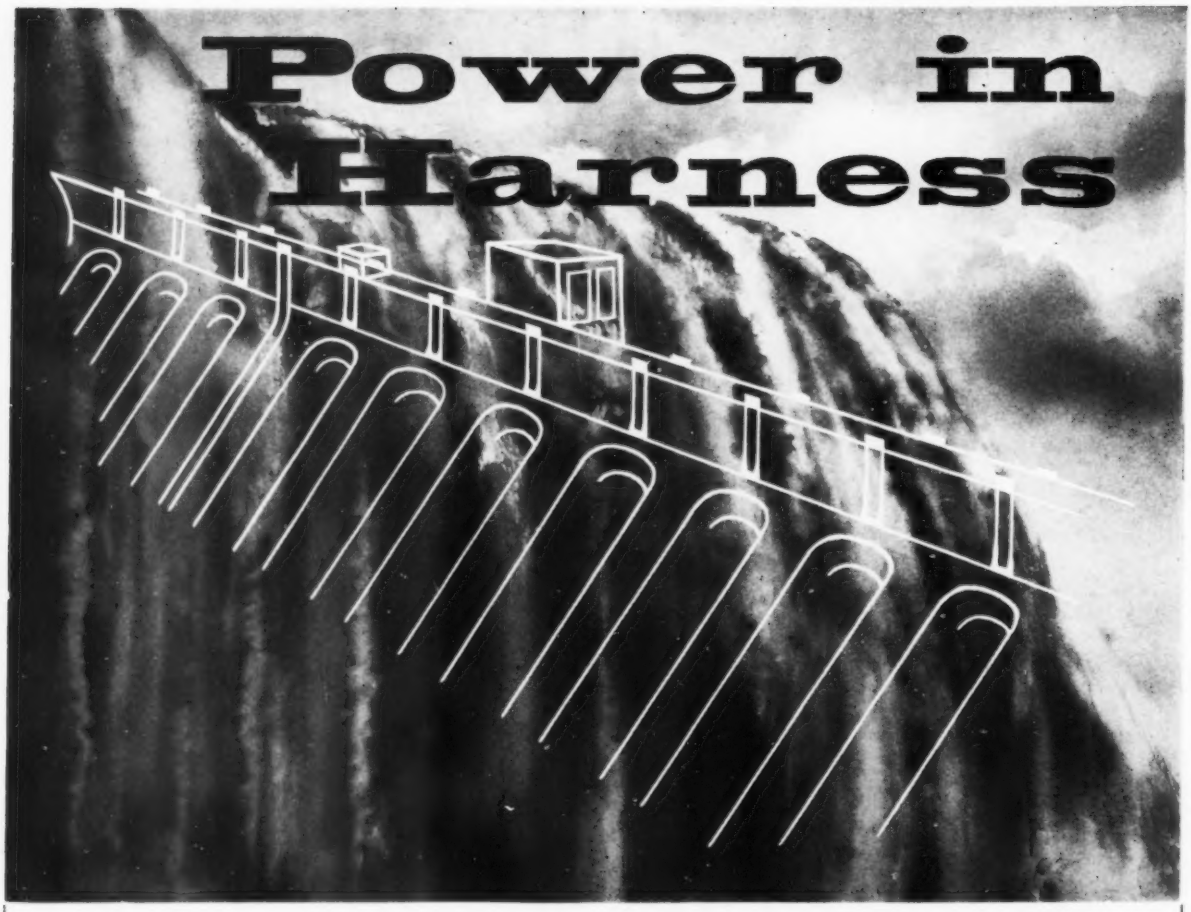

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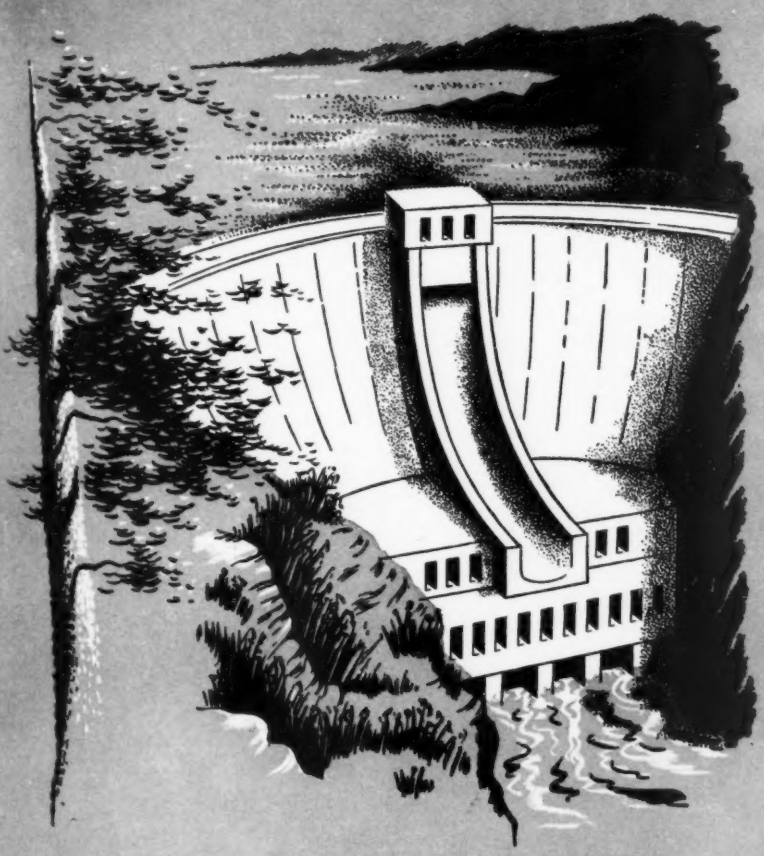

FERRANTI

Transformers for Hydro-Electric Schemes

As specialists in the manufacture of transformers for more than 65 years, FERRANTI LTD. have been associated with many of the world's largest hydro-electric schemes.

More recent achievements include contracts for over 4,000,000 kVA of large high voltage power transformers, among which are :-

CENTRAL AFRICA

Two 120,000 kVA and ten 60,000 kVA 330,000 volts three-phase Transformers have been ordered by the Central African Federation Power Board for the Kariba Hydro Electric Scheme. This is the largest overseas transformer contract ever to be placed in the United Kingdom.

CANADA

Seven 71,000 kVA, 301,400 volt single-phase Ferranti generator transformers and six 37,000 kVA, 275,000 volt step-down Ferranti transformers have been ordered by the Aluminum Company of Canada Ltd. for the Alcan Project at Kemano and Kitimat, British Columbia. Four of the generator transformers and all of the supply transformers are now in commission.

U.S.A.

GARRISON DAM PROJECT Nine 33,333 kVA, 230,000 volt single-phase Ferranti transformers forming three 100,000 kVA banks have been supplied to the U.S. Army Corps of Engineers for the Garrison Dam Project, North Dakota.

McNARY DAM PROJECT Six 56,000 kVA, 230,000 volt, single-phase transformers have been ordered by the U.S. Army Corps of Engineers for the McNary Dam Project, Oregon.

DALLES DAM PROJECT The contract covers eighteen 63,000 kVA, 230 kV and three 63,000 kVA, 115 kV, single-phase transformers for the U.S. Army Corps of Engineers Dalles Dam Project, Columbia River.

NEW ZEALAND

Twenty-six Ferranti large power transformers have been ordered by the State Hydro-Electric Department of New Zealand. Thirteen 14,800 kVA, single-phase, 50 cycles, 11/220 kV transformers have been supplied to Roxburgh, and thirteen 9,259 kVA, single-phase, 50 cycles, 11/220 kV transformers have been supplied to Whakamaru.

INDIA

Three Ferranti 20,000 kVA, OFB, 3-phase, 50 cycles, 132/33-66 kV transformers with on-load tap changing gear and two Ferranti 10,000 kVA, ON/OB, 3-phase, 50 cycles, 132/33 kV transformers with on-load tap changing gear have been supplied for the Damodar Valley Power Scheme in India.

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YORKSHIRE *transformers*

serve the power supplies of Great Britain

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Tel: Dewsbury 1691-2 Grams: Transforma, Dewsbury

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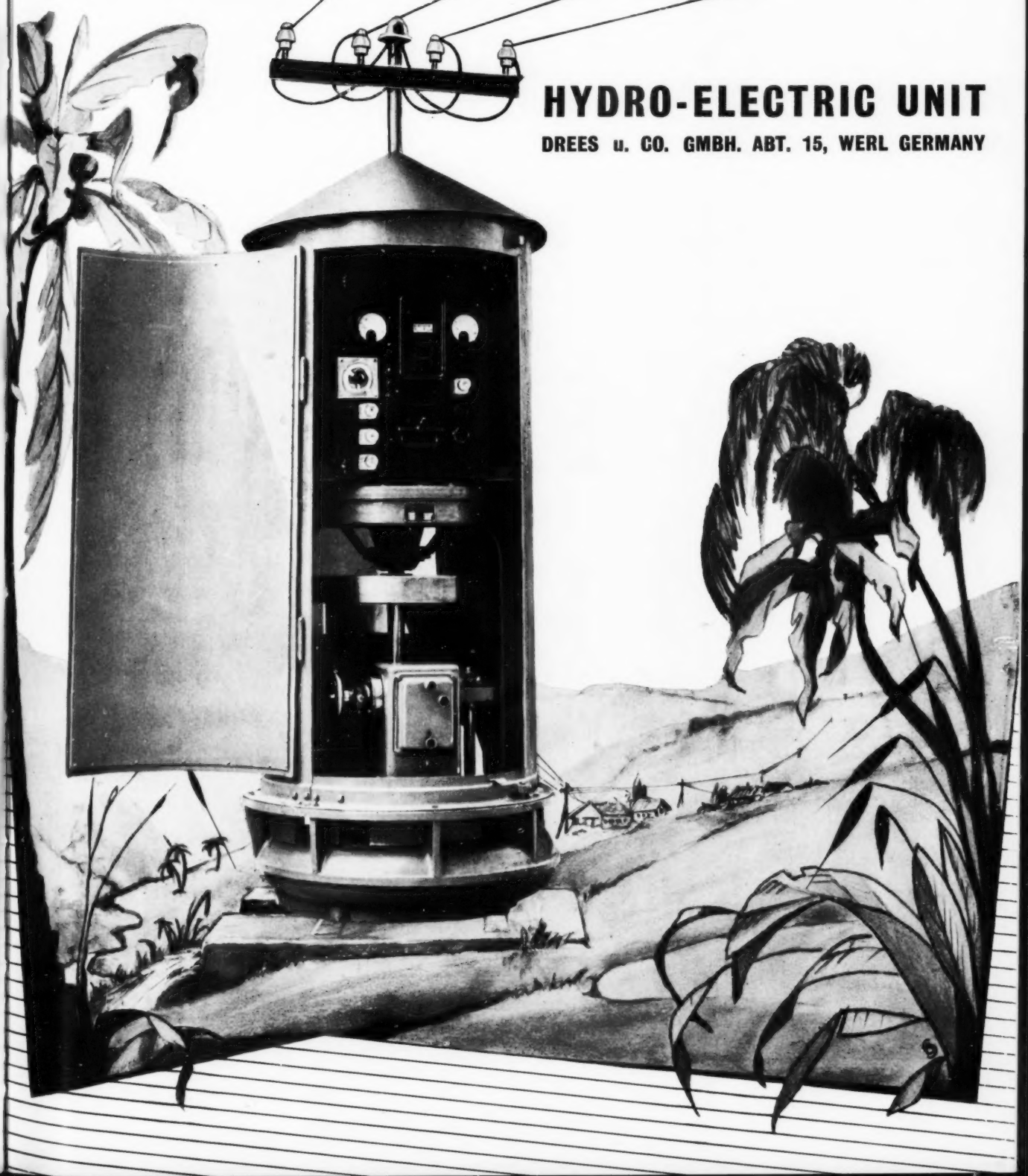

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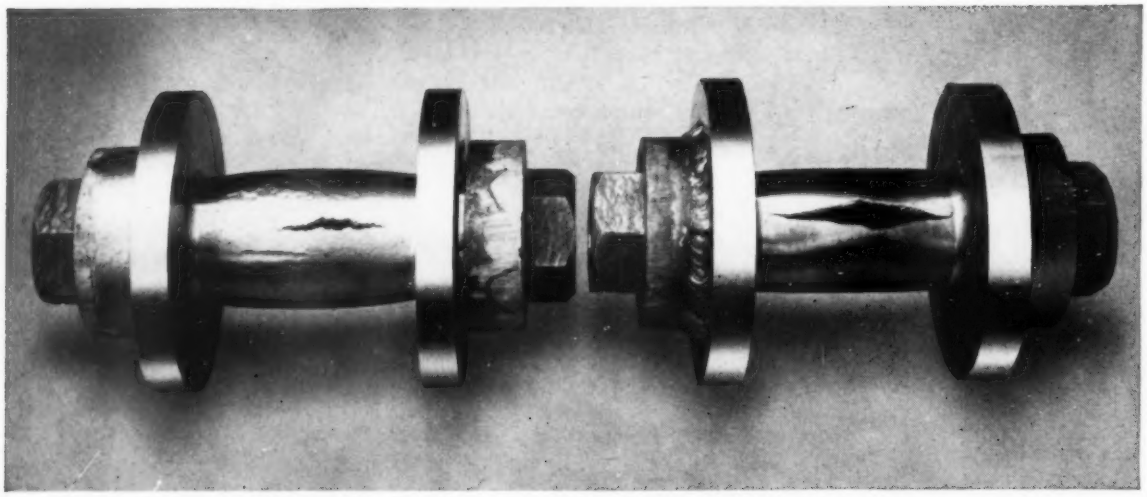

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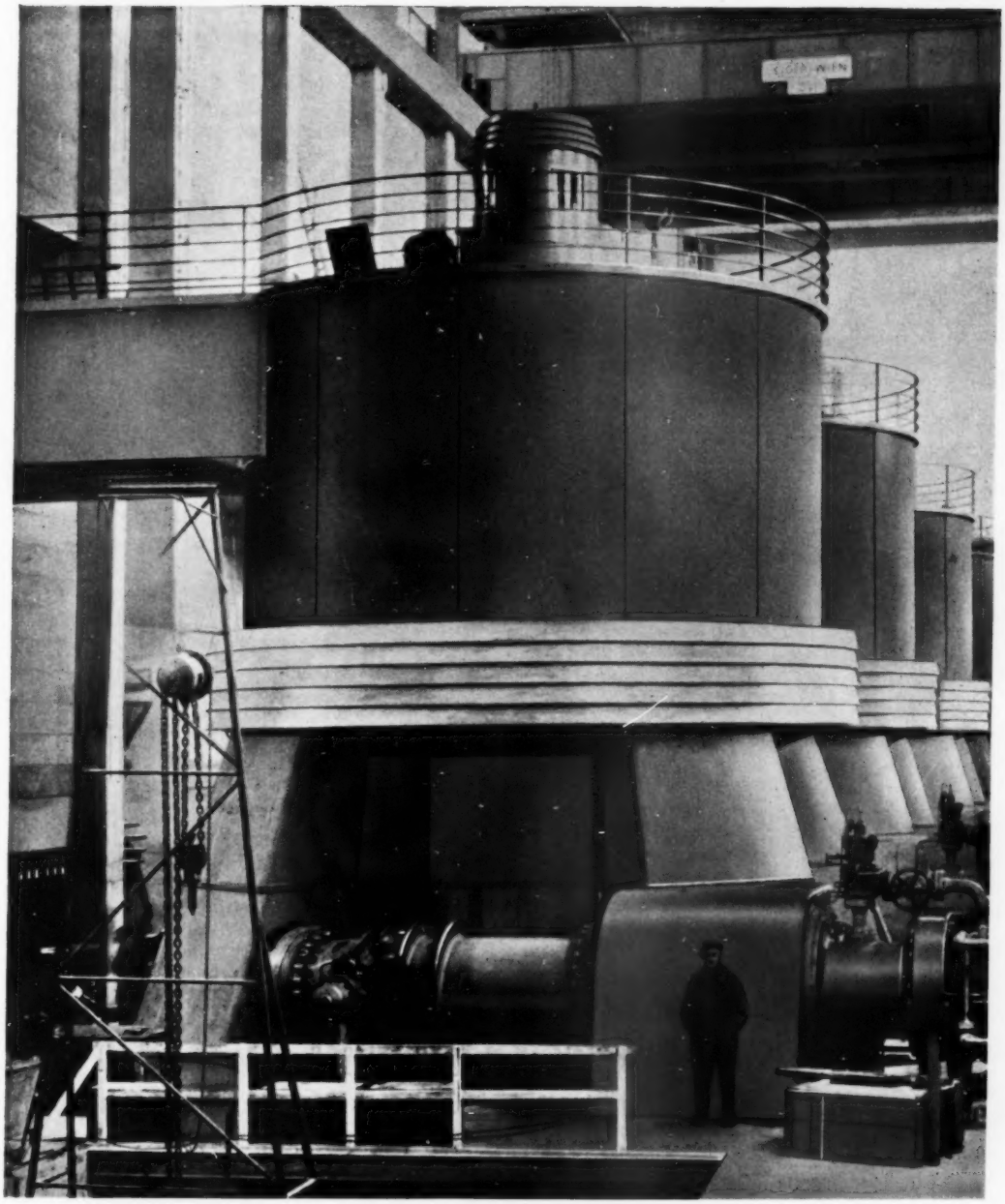

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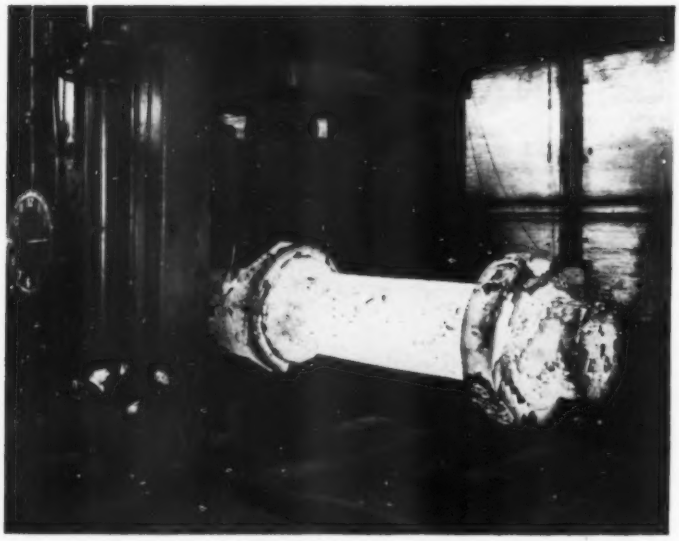

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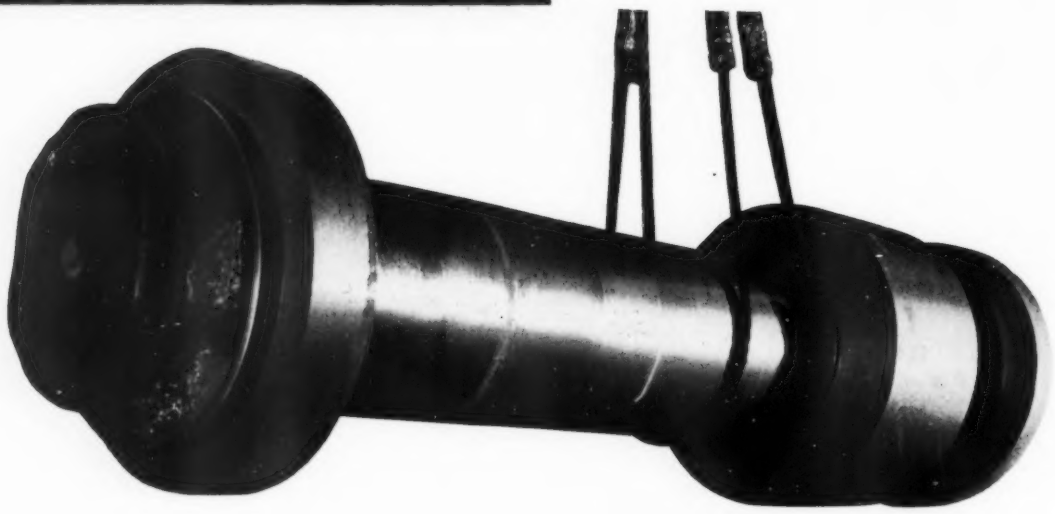

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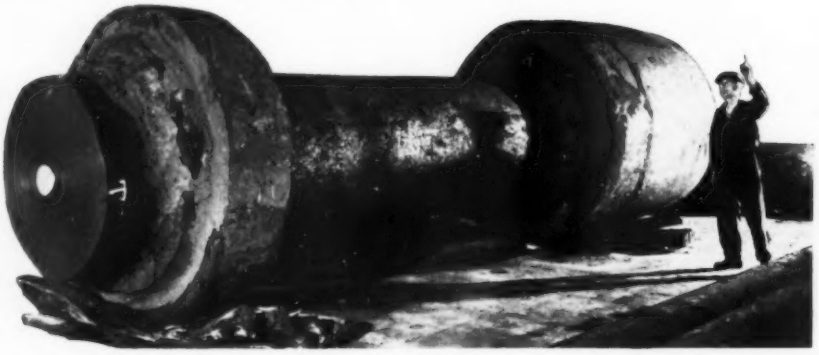

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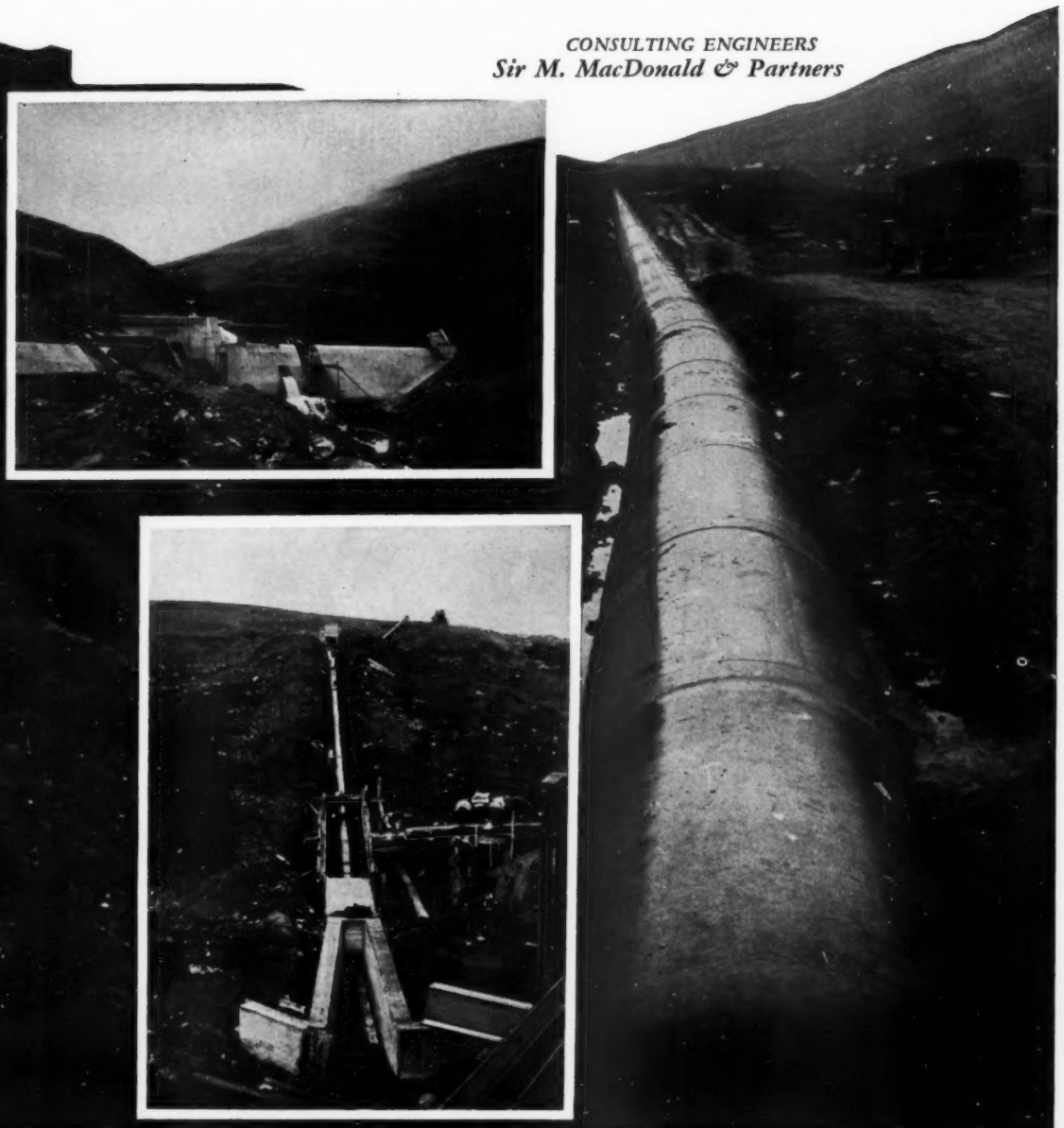

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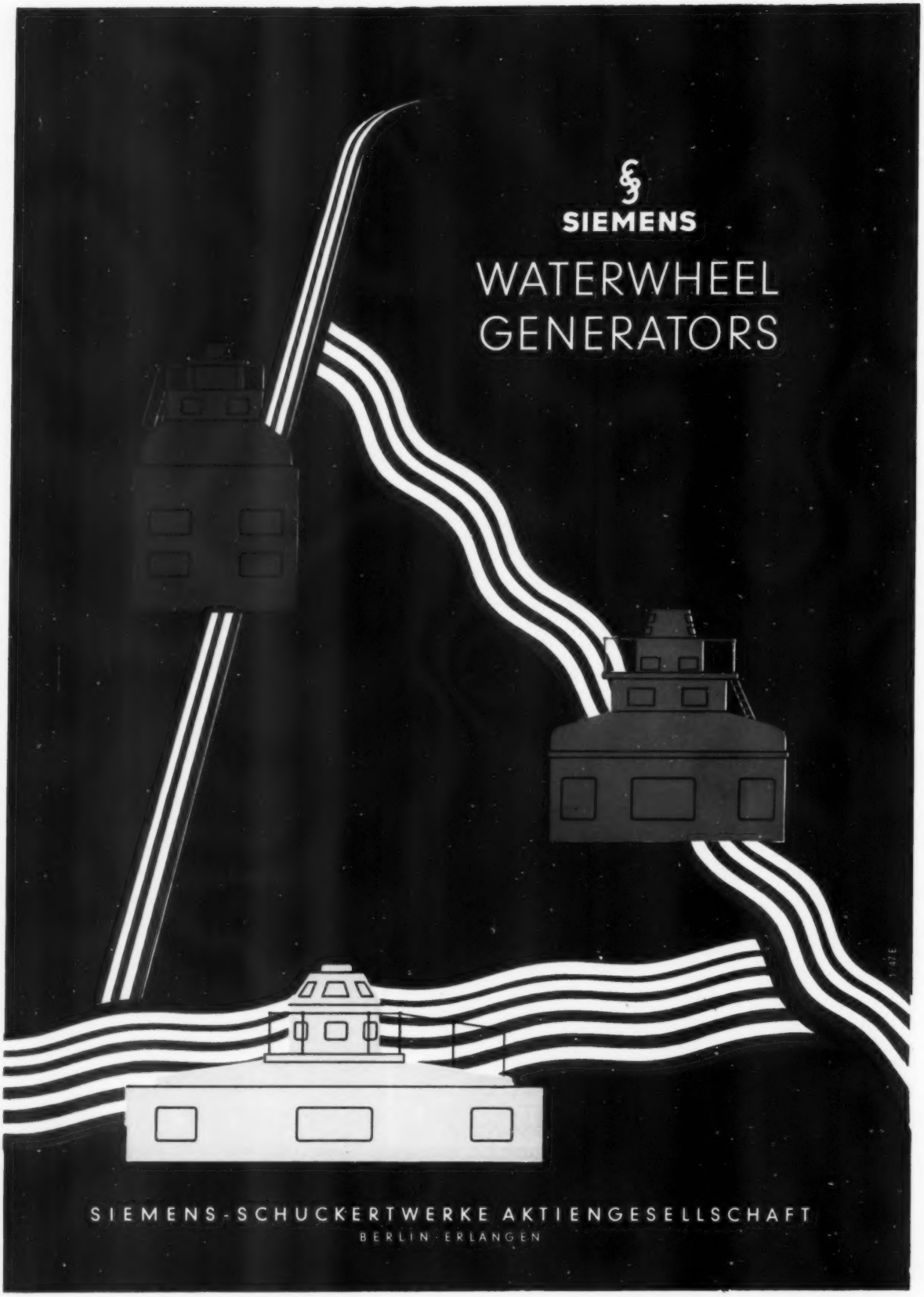
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Editor: H. STOREY, M.C., B.Sc., M.I.M.E.

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Gaps in Knowledge

ALTHOUGH the accounts published in this and other journals describing major hydro-electric projects are mostly a record of man's triumph over natural difficulties, nevertheless from time to time we do hear of water-power schemes that give trouble. Leakages occur, for instance, and foundations become undermined. A European dam gave cause for anxiety during April, and it is expected that six months will be required for repairs to the damage caused by undermining. In any undertaking of this challenging type, difficulties and occasional failures are inevitable; what is important is that all hydro-electric engineers should be enabled to learn valuable lessons from these rare mistakes.

The challenge facing any civil engineer designing a dam is still as breathtaking in scale as it ever was, in spite of the additional means provided by science for the evaluation of the qualities of the site. Take, for example, the well-known dam at Ghrib in Algeria where reasons connected with irrigation and other matters made it necessary to build a dam on ground which no engineer would ever have chosen. The site was extremely poor from the foundation point of view, and very heavy settlements had to be envisaged. The surrounding ground was far from watertight. Twenty years ago, when the project was completed, the engineers decided that as they had to build a dam on this site they would use a watertight screen of bituminous concrete, covered by an outer mask of porous concrete to prevent the heat of the Algerian sun from causing it to melt and flow. The work was brilliantly successful; but when in recent years the screen was inspected, it was found that the forward mask of reinforced porous concrete had not really been necessary, and that a layer of reflecting paint would have been sufficient. This valuable information, made available freely, has doubtless saved, in the aggregate, vast sums of money.

The challenge represented by the unknown nature of the ground on which a great structure is to be built must always remain formidable. The fact that the majority of hydro-electric dams encounter no trouble of any kind during their unlimited working lives is sufficient proof of the high degree of skill used for it to be unnecessary to pretend that there are no occasional failures. No layman, intelligent enough to realise the nature of the challenge and the forces encountered (such as underground springs, corrosive thermal nappes, hidden faults in sound rock, to mention only a few) could ever dare to point a finger of criticism at the designers of a major project on which some troubles occurred in the course of its early life. We therefore venture to put forward a plea for more information about the inevitable minor difficulties encountered in the operation of hydro-electric projects.

There are, of course, many cases where information about the causes of trouble is made available, but in our travels we hear, from time to time, of other cases. To mention a typical instance, the injecting of the ground in the neighbourhood of a major structure is carried out with various types of grout, and in some cases, after the lapse of a few years, it has been found necessary to re-inject to preserve watertightness; and it has been the practice to drill test borings, to find out from the cores thus obtained how the original grout had behaved and whether the action of time and of substances in the ground had altered its characteristics. We have known of cases where it has been felt, wrongly in our view, that the necessary re-injection cast a reflection in some way on those who originally designed the project; the suggestion has been that the original grouting was not planned sufficiently well, and in consequence further work had to be done some years later. This is a narrow viewpoint, and one which we are certain is not widely held; and yet the withholding of information on one particular project, for what is wrongly thought to be prestige reasons, may well mean that the engineers concerned with one of the few other projects in the world almost exactly similar in characteristics of terrain may be deprived of information that would save them from serious anxiety.

In the hydraulic, mechanical and electrical aspects

of hydro-electric work, all engineers start more or less on the same footing. The basic knowledge concerning the behaviour of the materials with which the mechanical engineers have to deal, or the electric currents which are the concern of the electrical engineer, is available to all, and the question of local abnormalities hardly enters into the design of the equipment in these departments. But in spite of the greater precision of design obtainable in these fields, the engineers concerned are faced with their appropriate problems of nature (cavitation, for example) and information on cases of undue wear and tear, unsatisfactory running, or mechanical or electrical breakdown, cannot fail to be of benefit to engineers in general. Nevertheless, the civil engineer is particularly vulnerable to intangibles. Equipped with the greatest possible knowledge of geology, soil mechanics and all other sciences bearing on the nature of the earth's crust, he has still to face many unknown factors, some of them revealing themselves only after the structure has been built.

Thus, once again we venture to reinforce the plea made earlier for the fullest possible degree of information to be made available about operating experience. Of course, the columns of this journal, the proceedings of the Conference on Large Dams (part of the World Power Conference organisation) and the proceedings of the civil-engineering institutions throughout the world do give a large amount of data on operating experience; but a prolonged study of much of this information shows that there are still gaps to be filled, and these gaps nearly all relate to the occasional minor failures which could act as pointers towards ensuring that similar circumstances would not occur on other projects.

New Dam for Persia

THE Persian Government have placed a contract with the Development and Resources Corporation of New York for completing the first stage of a series of multi-purpose projects in the Khuzestan region of south-west Persia. The first of these consists of the Dez dam which will have a finished height of 620 ft. and will impound a sufficient reserve of water to supply a generating station having a capacity of 520 MW and to irrigate 375,000 acres. Funds are being provided from the Persian oil revenues, 60% of which have been earmarked for economic and social development.

The Development and Resources Corporation was formed in 1955 by Mr. D. Lilienthal and Mr. G. Clapp, formerly of the Tennessee Valley Authority, and Lazard Brothers & Company.

Aluminium Limited

THE operations of Aluminium Limited for the year 1957 were marked by a decline in demand and a resulting fall in net income to $41·5 million, compared with $55·5 million for the previous year. During the year the Aluminum Company of Canada Limited produced 556,700 tons of primary aluminium as compared with 620,300 tons in 1956, part of this reduction being due to labour difficulties which involved a prolonged strike at the Arvida works.

Capital expenditure under the long-term expansion programme was $180 million, bringing the plant and facilities up to $1,300 million. The new capital was principally spent in providing additional hydro-electric power and better facilities for producing raw materials. Of this $51 million was expended in Quebec, mainly on the Chute-des-Passes hydro project, which is scheduled for operation in 1959, $51 million in British Columbia, for additional power and smelter facilities at Kitimat, and $44 million on bauxite developments in Jamaica, British Guiana and French West Africa.

New Power Expansion in Mexico

WITH the inauguration of the new hydro-electric plant at Tingambato, Mexico, recently, the country attains an installed capacity of 2,210,986 kW, representing an increase of over 10% in respect to the preceding year.

Since its establishment on December 1, 1952, the Federal Electricity Commission of Mexico has put into operation plants with a total capacity of 333,499 kW. The plant inaugurated in Mexico during this five-year period, which includes Tingambato and the second units at Oviachic and Dos Bocas, totals 514,938 kW. This has been the period of greatest national electrification made by the Federal Government. It should be remembered that the works at Tingambato were commenced during the previous six-year period.

The FEC system, with Tingambato, has now a capacity of 352,000 kW and connected to the Mexican Light & Power Company's system, attains a total of 843,000 kW. This interconnected system of FEC and ML&P represents 48% of the capacity destined for public service of the country, and will attain 53% within the near future, once the new unit at Lecheria has been completed.

The Ogoochi Dam, Japan

THE construction of the Ogoochi dam was first projected as long ago as 1938 but was held up by the war and other difficulties and the scheme was not completed until November 1957. It consists of a straight gravity structure 149 m. high and having a crest length of 345 m. The thickness at the crest is 9 m. and at the bottom 131 m., the total volume of concrete used being about 1·7 million cu. m. It provides a maximum storage capacity of 191 million cu. m., the resulting lake having an area of 4·36 sq. km. and a length of 13·5 km. The associated downstream power stations will have a capacity of 70 MW.

Progress at Brownlee

SOME 2,000 men are now being employed in the construction of the Brownlee project on Snake River, Idaho, and it is hoped that power generation will begin in September. This means that a period of only 33 months will have elapsed since the work began, which must be regarded as a very creditable performance in a scheme of this magnitude. The rockfill dam will contain nearly six million cu. yards of rock and 430,000 cu. yards of clay and is now more than 80% completed. It is about a third of a mile thick at the base and rises up to a height of 400 ft. above the deepest foundation. The initial generating capacity will be 360,400 kW and it forms part of a larger development scheme on the Snake River which will ulti-

mately support a capacity of nearly 1¼ million kW.

The Brownlee development is being constructed by the Morrison-Knudsen Co. Inc. for the Idaho Power Company. Fuller technical details were given in our April 1957 issue, page 138.

Developments on the Douro

ENCOURAGING progress in the construction of the three Portuguese developments on the international stretch of the Douro is recorded in the 1957 Report of Hidro-Eléctrica do Douro. The most forward of these developments is at Picote, where the first of the three 60 MW Francis sets revolved for the first time on December 19, 1957. Erection of the second set had reached the alternator, and the turbine of the third set was in course of installation. Concreting of the body of the dam had been completed, and most of the civil engineering had reached the finishing-off stage. Two of the three 15/220 kV main transformer banks had been placed in position, and the remaining bank was in course of installation. In the outdoor switching station all the main equipment had been installed. A 220 kV line to Coimbra was in service and a similar line to Oporto was under construction.

At Miranda the diversion tunnel, 330 m. long and 93 sq. m. in section, had been driven and was being concrete lined, and excavation of the underground machine hall was in progress.

Preliminary work had also been carried out at Bemposta.

Symposium on Rockfill Dams

THE American Society of Civil Engineers is sponsoring a Symposium on Rockfill Dams at its Portland, Oregon, meeting of June 23–27, 1958. Seventeen papers will be presented, which will bring up to date the experience with most of the existing rockfill dams in the world which are 200 ft. or more in height. For the purposes of the Symposium a rockfill dam is considered to be one that relies on dumped rock as a major structural element, and such dams are classified according to the type of impervious element employed, viz., impervious face membranes, sloping earth cores, thin central cores and thick central cores.

Tidal Energy

A PUBLICATION issued by the United Nations, Department of Economic and Social Affairs, New York, and entitled "New Sources of Energy and Economic Development" reviews the present state of knowledge in regard to solar energy, wind power, geothermic energy, tidal energy and thermal energy from the sea. The section on tidal energy arrives at rather disappointing conclusions as there appear to be few sites which are suitable for large-scale power development. These conclusions are based on the supposition that a site is hardly worth considering unless its annual output is at least several billion kWh and this, of course, adds to the difficulty of integrating such a large scheme in a suitable grid system. In what appears to be a puzzling contradiction to this finding, the report recommends that studies should be made of small tidal installations in coastal countries which now lack power resources. It suggests that in many cases basins of, say, 10 hectares in surface area could easily be closed off by means of inexpensive dykes to sustain a station of 1 MW capacity producing an output of a few million kWh per annum. Such stations, it is stated, could be operated either in isolation for such purposes as paper pulping, flour milling, etc., or in conjunction with diesel-driven pumps to improve the regularity of output.

Norwegian Water-Power Developments to Fight Slump?

A LARGE-SCALE development of Norwegian water-power resources, over and above the present harnessing schemes, as a possible countermeasure against an economic slump, was indicated recently by the Norwegian Minister of Industrial Affairs, Gustav Sjaastad. The extra electricity thus produced would be exported. The Minister pointed out that it was now technically possible to export large quantities of electricity. Norway, however, would not be able to finance such large developments herself. She would have to invite foreign capital, and in order to make this more attractive, the rather rigid legislation governing the investment of such capital in Norway might have to be amended. Such amendment would be necessary in any case if the European Free Trade Area came into force.

Present hydro-electric installations in Norway have a capacity of 2,700 million kWh, which when the current four-year plan is fulfilled is expected to increase to 3,400 million kWh. This figure represents less than one-third of the total medium-priced water power which can be harnessed in Norway. Even if Norway could raise the necessary money to finance the development of these resources herself, it would be extremely difficult also to finance any industry which could utilise the extra power. Therefore, according to Hr. Sjaastad, the nation will have to face the question whether it should invite foreign capital.

Iowa Hydraulics Conference

THE Iowa Institute of Hydraulic Research is organising its Seventh Hydraulics Conference, which will be held on June 16–18, 1958. The theme of the conference will be prototype verification of experimental and theoretical prediction in hydraulics and related fields. Fourteen speakers from various parts of the United States will present papers during the five technical sessions, the sixth session being devoted to an inspection of the laboratories. A special exhibit is being arranged by the university library of its new History of Hydraulics collection. It will be recollected that the Institute recently published a book entitled "History of Hydraulics," by Hunter Rouse and Simon Ince, which was reviewed in our March issue.

Furnas Contract Awarded

GEORGE WIMPEY AND CO. LTD. have been awarded a contract worth the equivalent of more than £12 million for the construction of the Furnas hydro-electric project in Brazil. The contract is for the construction of an earth and rock-fill dam, together with a power station. The dam will be 400 ft. in height and located on the Rio Grande, 240 miles west of Rio de Janeiro.

The Correlation of Nuclear, Thermal and Pumped-Storage Capacity

This article represents an original contribution to the discussion of system load-factor analysis and the economics of pumped storage, with special reference to nuclear-energy generation

By DR. CHARLES JAEGER

PART ONE

THE purpose of this article is primarily to analyse the main factors in the economics of combined pumped-storage and nuclear energy. This enquiry led to some additional research on the more general problem of system load-factor analysis, which is included. It soon became obvious that the particular problem the writer intended to analyse, forcibly led to a more general enquiry on the theory of the system load factor.

In recent years a few papers and pamphlets have dealt with the economics of combined pumped-storage and thermal energy. In addition, authorities in charge of large combined electric systems, where pumped-storage may be introduced in the future, have used calculating machines for solving particular problems. The " answer " given by a calculating machine depends on which parameters have been fed into the computer and how they have been fed into it. Is not some simple basic thinking possible relying on more elementary mathematical analysis? A simplified mathematical approach may not necessarily be misleading; it may be helpful in clearing up important points and showing to what extent a result can be claimed to be general or not. Most papers published in Great Britain implicitly refer to the English system or to the British grid.[1,2] Much of the thinking is based on the existence in this system of modern highly efficient steam stations and of older, less-efficient low-pressure power stations. It is shown that new steam stations " improve the system efficiency " and when put into figures this gives to new steam stations an advantage over pumped storage. The question of similar system improvements caused by pumped storage is not analysed and are not figured in these publications.

On the other hand, British enquiries dealing specifically with " practical applications in Great Britain " have forcibly described storage schemes as visualised in this country, which are mainly of the " true or pure storage type " and the great variety of physical solutions, possible in other countries, have been somewhat disregarded.

Some German authors had a somewhat different approach, inspired by the earlier publications of Prof. Musil,[3] the well-known teacher at the Graz Institute of Technology. Hydro-power is a main asset in Bavaria,[4,5] where many of the existing pumped-storage schemes have been built. Nevertheless the German authors choose to compare pumped storage with thermal energy and not with hydro-power so that the methods they use and the figures at which they arrive are comparable with those of their British counterparts.

When starting his analysis of the economics of pumped storage combined with nuclear power, the writer was aware of the relationship between the particular problem he wanted to investigate and the approach to pumped storage and steam by others. The new approach will show the problem of the " improved system efficiency " in a new light.

Before reaching the core of the problem, some preliminary remarks concerning pumped-storage schemes may be helpful. The main types of pumped storage have been described in recent papers.[6] " Pure or true pumped storage " is a system where the total volume of water discharged and the gross head are the same for the pumping station and for the turbines. In " mixed pumped storage " schemes the turbines will run either with a greater head and/or with a larger total capacity than the pumps so that the produced energy may be greater than the absorbed energy, even taking into account the efficiency factor of the pumps and turbines. Pumped storage visualised in Great Britain is mainly of the pure or true type (examples: Ffestiniog, Balmaha). The extended Loch Sloy scheme is more of an exception, being of the mixed type, as the existing hydro-power scheme would be extended by creating several powerful underground pumping stations.

The economics of a particular pumped-storage scheme obviously depend on the type of the scheme. Another factor of great importance is the head. It can be shown that the storage volume required for a pumping scheme is inversely proportional to the available gross head. This fact has a direct bearing on the cost of pumped-storage energy, as seen in Fig. 1 where the capital cost per installed kW. has been analysed for several German schemes built

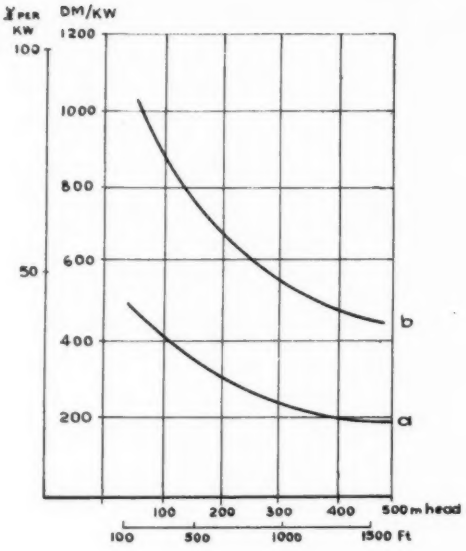


FIG. 1.

Cost of German pumped storage stations in DM or £ per KW for heads from 100 to 1500 Ft.
a) Costs in 1926 - 30
b) Costs in 1956 average.

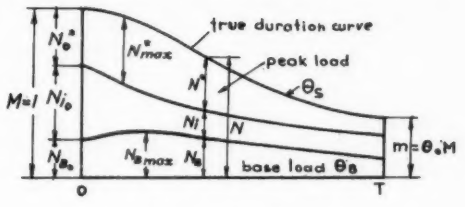


FIG. 3.

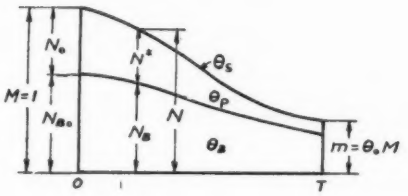


FIG. 4.

1957 SYSTEM LOAD CURVES FOR ENGLAND AREAS.

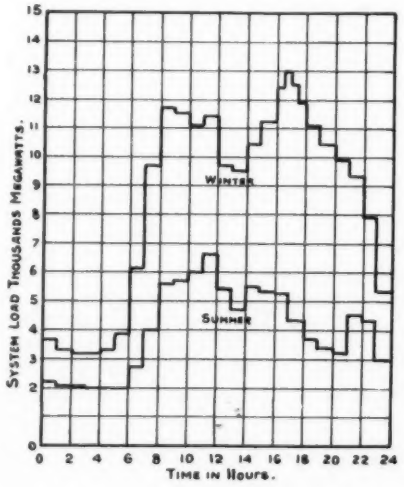


FIG. 2a.

1957 LOAD DURATION CURVE FOR ENGLAND AREAS.

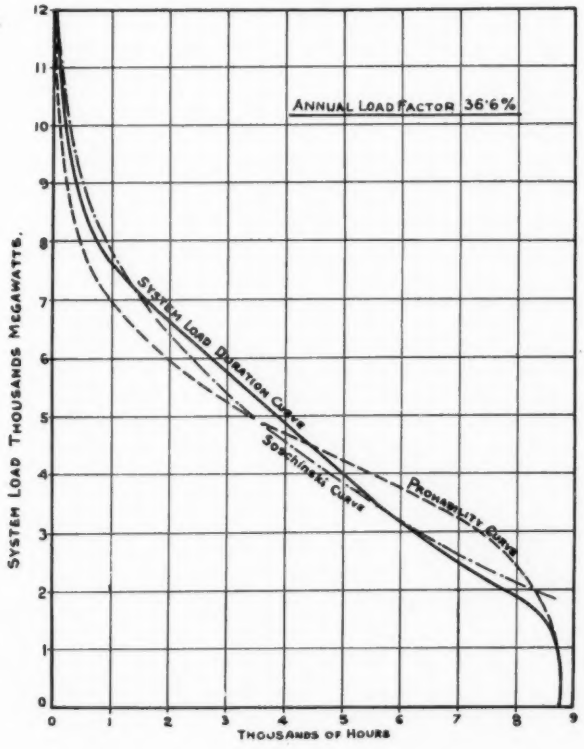


FIG. 2b.

between 1926 and 1930 (transmission cost being neglected). The curves have then been adjusted to 1956 prices in Germany.[5] Turbine efficiency, pump efficiency and friction losses in the pipes (conduit efficiency) are the other factors on which the economics of the plant depend.[6]

When dealing with the problem on more general lines these vast differences between possible schemes must be kept closely in mind.

General system load-factor analysis

Let us assume that an electric generating system has a load factor θ_s. M is the maximum load to be faced over a period T and $m = \theta_0 M$ is the minimum load over the same period. The chronological diagram giving the variation of the load N over the whole period T can always be used to trace a duration curve over the same basic period T (e.g. $T = 1$ day or 1 week or 1 year). It will also be assumed that this load-duration curve can be represented by a suitable mathematical function which will be used for further analysis.

In most cases the energy is produced by stations of different characteristics; some of them will be classified as base-load stations with a high individual load factor, the others as peak-load stations with a low load factor. The area under the chronological curve $N = N(t)$ (t = time) will be divided accordingly into partial areas, which can be used to subdivide the area under the duration curve (Fig. 2). The main contour line is supposed to be a true duration curve. In the more general case the intermediary curves so obtained are not necessarily "true" duration curves, but they often will be nearly such. It will be shown that this is not an essential point for the discussion.

(*a*) *Case of load system being divided in two load areas only.* The load curve for the whole system is supposed to be represented by a duration curve over a period T (Figs. 3 and 4) with $0 < t < T$ and t being a time abscissa. The maximum load on the system is $M = 1$ and this occurs for the time duration $t = 0$. N represents any ordinate corresponding to a duration t. The area below the duration curve is divided into two so that in $N = N_B + N_p$, N_B represents a base load and N_p a peakload. For $t = 0$, $M = N_{B0} + N_{p0}$. The maximum of N_p is $N_{p\max}$. Most probably, if N_B really represents a base load, $N_{p\max}$ occurs at the time $t = 0$ and $N_{p\max} = N_{p0}$. A similar remark is valid for $N_{B\max}$ and N_{B0}. An integral like $\int_0^t N\,dt$ represents an energy.

Three load factors θ_s, θ_B and θ_p can be defined for the loads.

θ_s for the whole system is

$$\theta_s = \frac{1}{MT}\int_0^T N\,dt$$

For the two partial loads we have

$$\theta_B = \frac{1}{N_{B\max}T}\int_0^T N_B\,dt \quad \text{and} \quad \theta_p = \frac{1}{N_{p\max}T}\int_0^T N_p\,dt$$

Let us write explicitly this load factor θ_p:

$$\theta_p = \frac{1}{N_{p\max}T}\int_0^T (N - N_B)\,dt$$

$$= \frac{M}{N_{p\max}}\cdot\frac{1}{MT}\int_0^T N\,dt - \frac{N_{B\max}}{N_{p\max}}\frac{1}{N_{B\max}T}\int_0^T N_B\,dt$$

$$\theta_p = \frac{M}{N_{p\max}}\theta_s - \frac{N_{B\max}}{N_{p\max}}\theta_B \qquad \ldots(1)$$

This expression can be shown to be still valid in

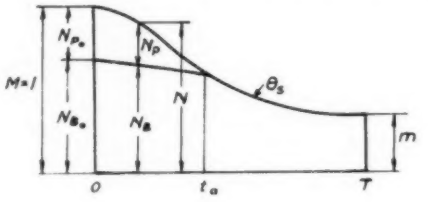


FIG. 5.

the case of Fig. 5, where the ordinate N_B becomes equal N for $t \geqslant t_a (N = N_B$ for $t \geqslant t_a)$.
Then

$$\theta_p = \frac{1}{N_{p\max}T}\int_0^{t_a} N_p\,dt = \frac{1}{N_{p\max}T}\int_0^{t_a} (N - N_B)\,dt$$

$$= \frac{1}{N_{p\max}T}\left[\int_0^{t_a} N\,dt + \int_{t_a}^{T} N\,dt - \int_0^{t_a} N_B\,dt - \int_{t_a}^{T} N\,dt\right]$$

$$= \frac{M}{N_{p\max}}\frac{1}{MT}\int_0^T N\,dt -$$

$$- \frac{N_{B\max}}{N_{p\max}}\frac{1}{N_{B\max}T}\left[\int_0^{t_a} N_B\,dt + \int_{t_a}^{T} N\,dt\right]$$

which obviously yields again

$$\theta_p = \frac{M}{N_{p\max}}\theta_s - \frac{N_{B\max}}{N_{p\max}}\theta_B \qquad \ldots(1)$$

This is a very simple linear relation between θ_s, θ_p and θ_B. As the whole problem deals with simple integration of areas, a linear equation was to be expected. Formula (1) can easily be discussed if the two parameters $M/N_{p\max}$ and $N_{B\max}/N_{p\max}$ the meaning of which is obvious, are known.

In most cases $N_{B\max} = N_{B0}$ and $N_{p\max} = N_{p0} = M - N_{B0}$
With $M = 1$, then:

$$\frac{\theta_p}{\theta_B} = \frac{1}{1 - N_{B0}}\frac{\theta_s}{\theta_B} - \frac{N_{B0}}{1 - N_{B0}} \qquad \ldots(1a)$$

If we take θ_p/θ_B and N_{B0} as main variables, θ_s/θ_B being a free parameter, then this linear relation between θ_p/θ_B and θ_s/θ_B is represented by a family of hyperbolic curves shown in Fig. 6. It is easy to see that

$$\theta_p/\theta_B = 0$$

when $N_{B0} = \theta_s/\theta_B$ and that for $N_{B0} = 0$

$$\theta_p/\theta_B = \theta_s/\theta_B$$

θ_p/θ_B becomes infinite for $N_B = 1$.

For N_{B0} to be positive we must have

$$N_{B0} = \frac{\theta_s/\theta_B - \theta_p/\theta_B}{1 - \theta_p/\theta_B} > 0$$

The curves in Fig. 6 show clearly that only cases where $\theta_s/\theta_B < 1$ have to be considered, that is

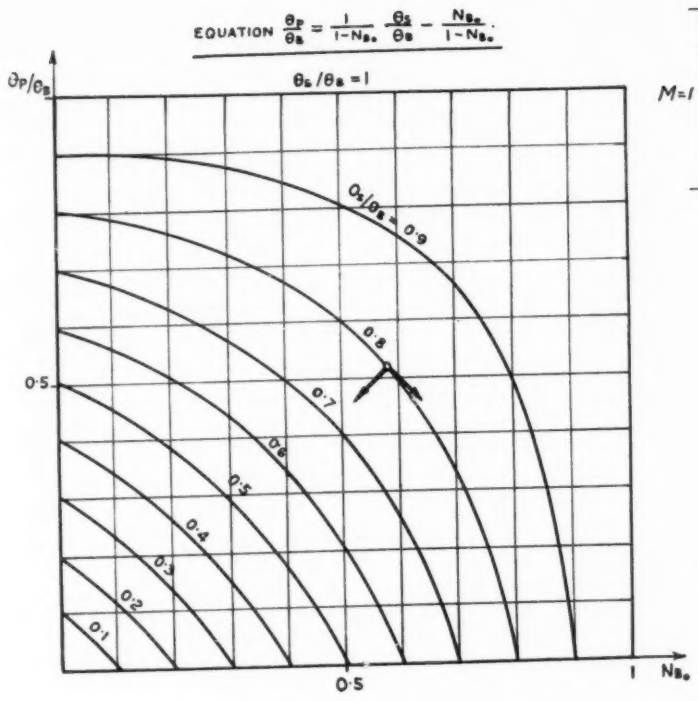


FIG. 6.

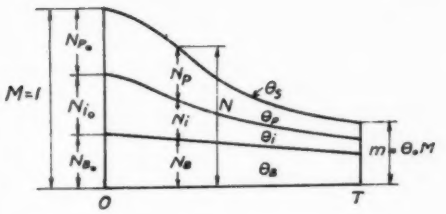


FIG. 8

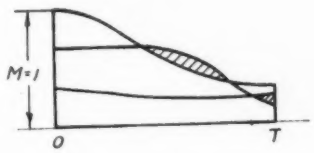


FIG 9

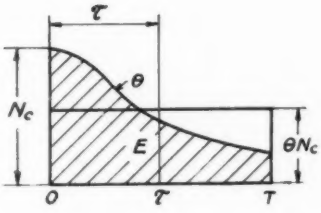


FIG. 10.

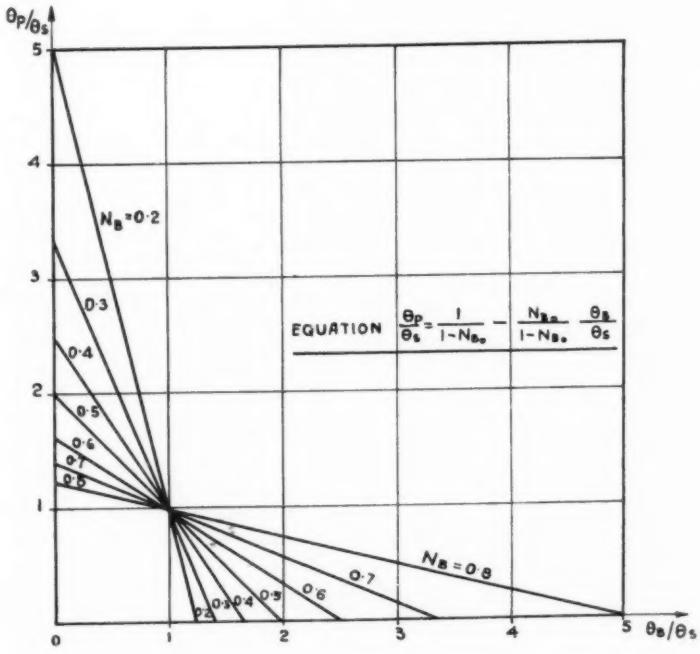


FIG. 7.

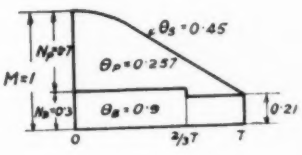


FIG.11

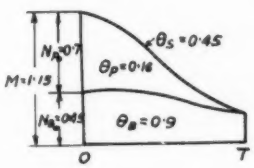


FIG.12.

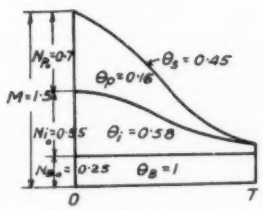


FIG.13.

$\theta_B < \theta_s$, which corresponds to the definition of a "base load." The case $\theta_s/\theta_B = 1$ or $\theta_B = \theta_s$ yields a horizontal line, as does $\theta_p = \theta_s = \theta_B$ also. Fig. 6 also shows when increasing the base load N_{B0} but remaining on the same curve $\theta_s/\theta_B = \text{const.}$ (that is maintaining the ratio of the load factor θ_s and θ_B constant) how the peak-load factor θ_p decreases and conditions for the peak load deteriorate. Similarly if the base-load factor θ_B is being raised, maintaining the system load factor θ_s constant, the conditions for peak load also deteriorate rapidly.

Fig. 7 shows conditions even better. This figure has been obtained in writing equation (1) like this:

$$\frac{\theta_p}{\theta_s} = \frac{1}{1-N_{B0}} - \frac{N_{B0}}{1-N_{B0}} \frac{\theta_B}{\theta_s}$$

which clearly shows how θ_p/θ depends linearly upon θ_B/θ_s when N_{B0} is maintained constant. Let us assume that θ_s is maintained constant. For $N_{B0} = 0{\cdot}5$ the decrease of θ_p is equal to the increase of θ_B because the straight line $N_{B0} = 0{\cdot}5$ is inclined at 45° (assuming the two co-ordinates to have the same scale). If $N_{B0} > 0{\cdot}5$ then the straight line $N_{B0} = \text{const.}$ has a steeper gradient than 45° and any increase of θ_B causes a larger drop of θ_p. For $N_{B0} < 0{\cdot}5$ conditions are reversed.

The diagrams Fig. 6 and Fig. 7 permit an immediate analysis of the variations occurring on a system if one of the variables θ_s, θ_B, θ_p or N_{B0} is being altered.

(*b*) *Case of a load system being divided in three or more load areas.* When analysing a load system, division of the whole area in three or more sub-areas may be required. Conditions prevailing then are shown in Fig. 8.

According to this figure, any ordinate N is split up as follows:

$$N = N_B + N_t + N_p$$

and for $t = 0$

$$M = N_{B0} + N_{t0} + N_{p0} = 1$$

The maxima of these ordinates are respectively $N_{B\max}$, $N_{t\max}$ and $N_{p\max}$. In most practical cases we shall assume for simplicity that $N_{B\max} = N_{B0}$, $N_{t\max} = N_{t0}$ and $N_{p\max} = N_{p0}$.

The definition of θ_p gives:

$$\theta_p = \frac{1}{N_{p\max} T} \int_0^T N_p \, dt$$

$$= \frac{1}{N_{p\max} T} \int_0^T [N - (N_t + N_B)] \, dt$$

$$= \frac{M}{N_{p\max}} \frac{1}{MT} \int_0^T N \, dt - \frac{N_{t\max}}{N_{p\max}} \frac{1}{N_{t\max} T} \times$$

$$\times \int_0^T N_t \, dt - \frac{N_{B\max}}{N_{p\max}} \frac{1}{N_{B\max} T} \int_0^T N_B \, dt$$

$$\theta_p = \frac{M}{N_{p\max}} \theta_s - \frac{N_{t\max}}{N_{p\max}} \theta_t - \frac{N_{B\max}}{N_{p\max}} \theta_B \qquad \ldots(2)$$

where obviously

$$\theta_t = \frac{1}{N_{t\max} T} \int_0^T N_t \, dt$$

Here again a very simple formula is obtained which is linear in θ_s, θ_p, θ_t, θ_B with $M/N_{p\max}$, $N_{t\max}/N_{p\max}$, $N_{B\max}/N_{p\max}$ as parameters. Obviously the formula could be generalised, making $i = 1, 2, 3 \ldots n$.

A simplified formula is obtained by making $M = 1$ and $M_{i\max} = M_{i0}$ etc.: Then

$$\theta_s = N_{B0}\theta_B + N_{t0}\theta_t + [1 - (N_{t0} + N_{B0})]\theta_p \qquad \ldots(2a)$$

This compares with formula (1a) for two areas. The number of parameters is now too large for a general discussion with curves, but any practical problem can be discussed by using numerical values.

Equations (1a) and (2a) are so simple that it is hardly believable they have never been established before. They most probably have been used by other authors for solving particular problems. The aim of this paper is to introduce a more general discussion.

In these discussions it is immaterial if T refers to a day ($T = 24$ hours), a week ($T = 168$ hours), or a year ($T = 8760$ hours). The method is the same if the duration curve is established for a day, a week or a month, or if the load variation to be analysed cover a whole year. For determining the size of a reservoir, a characteristic weekly (or daily) diagram will be used. For general problems of economics a yearly duration curve will have to be referred to.

Knowing θ_d = working day load factor and θ_e = week-end load factor, and N_d and N_e the corresponding maximum load on that day, then the weekly load factor θ_{we} is given by

$$5N_d\theta_d T + 2N_e\theta_e T = 7N_{we}\theta_{we} T$$

with $N_{we} = N_d$, and $T = 1$ day; so that:

$$\theta_{we} = \frac{5N_d\theta_d + 2N_e\theta_e}{7N_d}$$

Knowing θ_{we} for different periods of the year (spring, summer etc.) the yearly load factor will be

$$\theta_{\text{year}} = \frac{\sum_1^{52} N_i \theta_i T_w}{M \sum_1^{52} T_w}$$

($T_w = 1$ week)

Examples of Improved System Efficiency

The validity of the general theorems expressed by the formulae (1) and (2) cannot be questioned. As all the load factors are obtained by integration of areas, the order in which the ordinates N, N_B or N_p are classified is immaterial and the theorems are still valid if the duration curves are replaced by chronological load curves like the ones in Fig. 2. The discussion is easier with duration curves and this is why they have been introduced here for demonstration of the formulae (1) and (2) or (2a).

The theorems expressed by these formulae do not depend on the actual shape of the duration curves but only on θ_s, θ_B, θ_p and θ_t. The only condition to be observed is that the curves must not cut each other.

Conditions as shown in Fig. 9, with "negative" areas are inacceptable and this point has to be watched when dealing with practical cases.

The main problem which can be discussed with the help of equations (1) and (2) refers to the interdependence of θ_s, θ_B, θ_p, θ_t and the N_0 values.

British power engineers used to calculate the "improved system efficiency" which is the improvement of the overall fuel consumption, resulting from the addition of new high-efficiency plants to the existing system. This definition can refer only to a system based entirely on steam-generated power. There is a broader definition of the variations of the system efficiency, however, which can either improve or deteriorate by

the addition of new plants. This definition will be illustrated by examples.

The yearly cost C of producing an amount of energy E can be expressed by a formula of the type

$$C = mN_c + nE$$

In this formula N_c is the installed capacity and m and n are convenient price-reduction factors. The British analysis is based on the following. Suppose that the energy E_B refers to new stations with a low price-reduction factor n_B and energy E_p to old stations with a high factor n_p so that $nE = n_B E_B + n_p E_p$. The larger E_B the lower the price C/E per unit of energy sent to the grid. In this particular case n_B and n_p depend on the coal consumption, on the steam-turbine efficiency and on the coal price, and new stations " improve the system efficiency."

There is another approach to this problem, that is, when a station with installed capacity N_c is run constantly at full capacity during a utilisation time τ, in order to produce an energy $E = \int_0^T N\,dt$, then $\tau = E/N_c$ (Fig. 10). But the same energy $E = \int_0^T N\,dt$ can be produced at constant load θN_c over the whole period T as that

$$E = \tau N_c = \theta T N_c$$

where θ is a load factor and $\tau = \theta T$.

For $T = 1$, τ becomes $\tau = \theta$ and if θ is increased, so will be τ and the price conversion factor m will be lower, as m obviously depends on the time during which the station is utilised. This second aspect of the problem should not be neglected as it may overshadow the first one.

There is, therefore, a major interest in a complete and detailed analysis of the load factor θ of power stations, or groups of power stations, because this too has a direct bearing on cost of energy. It has already been shown in Fig. 6 and Fig. 7 how the load factor θ_B of one group of stations in a system can influence the load factor θ_p of the other group and *vice versa*. The " system efficiency " can be influenced for the better or the worse depending on the values of m and of n for the different groups of stations.

Some examples may illustrate this discussion.

CASE 1 (Fig. 11 and Fig. 12).

A system represented by Fig. 11 has a total installed capacity $M = 1$. The base-load capacity is $N_{B0} = 0{\cdot}3$ and the peak-load capacity $N_{p0} = 0{\cdot}7$. The load factor of the base load is $\theta_B = 0{\cdot}9$ (Fig. 11) and the system load factor is $\theta_s = 0{\cdot}45$. Formula (1) yields

$$\theta_p = \frac{1}{1-0{\cdot}3} \times 0{\cdot}45 - \frac{0{\cdot}3}{1-0{\cdot}3} \times 0{\cdot}9 = 0{\cdot}257$$

If after a period of years the base-load capacity N_{B0} has increased by 50%, to 0·45, with θ_B still being 0·9, and the peak-load capacity N_{p0} remaining the same, the total installed capacity will be $M = 1{\cdot}15$ with $\theta_s = 0{\cdot}45$ and

$$\theta_p = \frac{1{\cdot}15}{0{\cdot}7} \times 0{\cdot}45 - \frac{0{\cdot}45}{0{\cdot}7} \times 0{\cdot}9 = 0{\cdot}16 \quad \text{(Fig. 12)}$$

There is a sharp drop in the value of θ_p. In spite of this, there is an obvious advantage in increasing the base load, as n_B is low for highly efficient stations and m is low for old stations. When the correct values are introduced for the factors n_B, n_p, m_B, m_p the system efficiency will be shown to have increased.

This example shows how the introduction of new steam plant influences the system efficiency. It also shows how with two main groups of stations N_B and N_p, the problem can be fully discussed without knowing the real shape of the actual duration curve. The system load factor $\theta_s = 0{\cdot}45$ determines the whole problem.

CASE 2 (Fig. 13).

Consider the same system after n' years, as it expands. The system load factor is still $\theta_s = 0{\cdot}45$. The base load is now generated by nuclear power ($N_B = 0{\cdot}25$) and the steam-generated power now covers the intermediary area of the duration curve with $N_i = 0{\cdot}55$. Peak capacity is still $N_p = 0{\cdot}70$ (Fig. 13) and it is assumed that θ_p is maintained at $\theta_p = 0{\cdot}15$. M is now $M = 0{\cdot}25 + 0{\cdot}55 + 0{\cdot}7 = 1{\cdot}50$. Let us assume that the nuclear energy has a load factor $\theta_B = 1$. The load factor for efficient steam stations will be, according to formula (2),

$$\theta_i = \frac{1{\cdot}5}{0{\cdot}55} \times 0{\cdot}45 - \frac{0{\cdot}70}{0{\cdot}55} 0{\cdot}15 - \frac{0{\cdot}25}{0{\cdot}55} \times 1$$
$$= 1{\cdot}225 - 0{\cdot}645 = 0{\cdot}580$$

Even reducing θ_p from 0·257 to 0·15, θ_i for steam would drop from 0·9 to 0·58 which may be undesirable. Either θ_p or θ_B must be modified to get a more balanced generation of power from different sources.

A better solution may be obtained if the nuclear power stations are run at a lower load factor say, $\theta_B = 0{\cdot}70$. Then

$$\theta_i = \frac{1{\cdot}5 \times 0{\cdot}045}{0{\cdot}55} - \frac{0{\cdot}70}{0{\cdot}55} \times 0{\cdot}15 - \frac{0{\cdot}25}{0{\cdot}55} \times 0{\cdot}70$$
$$= 1{\cdot}225 - 0{\cdot}51 = 0{\cdot}715$$

This solution in this last example puts nuclear energy on the same footing as thermal power the load factor of which is now reduced from 0·9 to 0·715.

Some authors have expressed the opinion that nuclear energy will be run at full load ($\theta_B = 1$) until the installed nuclear capacity is higher than the minimum night load on the system. This opinion is not likely to be correct because it would cause the load factor of steam stations to drop to a point where the system efficiency—as defined here on a very broad basis—would deteriorate. Nuclear energy will have to be balanced by pumped storage long before nuclear capacity reaches the minimum load N_{min} on the grid. A more general discussion of the problem of subdivision in three areas will be introduced later.

This second case too has been solved without knowing the exact shape of the duration curve. As will be seen, a more detailed analysis of the problem required a more precise knowledge of the system and, in particular, the shape of the duration curve.

The general problem next to be analysed can be summarised as follows: Consider a new base-load station (possibly nuclear power) which it to be installed and linked direct to a new peak-load station (pumped storage) so that the two stations combined—considered as a single unit—have a total capacity N_{B0} and a predetermined combined load factor $\theta_B < 1$. The problem is to design the peak-load station (pumped storage) so that the base-load station (nuclear power) is utilised at full capacity (θ nuclear $= 1$) or near to full capacity.

This second problem requires the knowledge of the shape of the duration curve.

(To be continued)

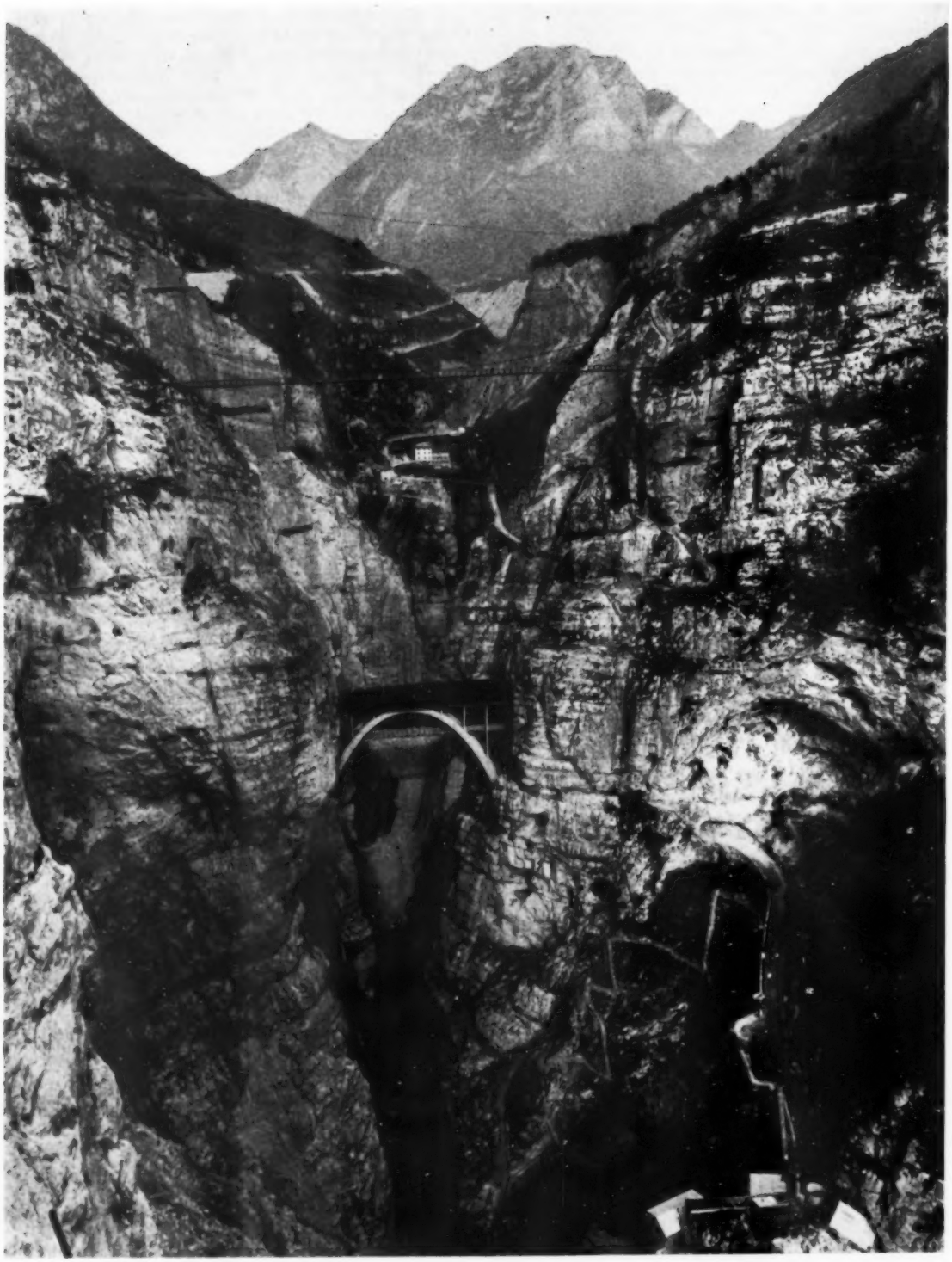


Fig. 33. A view of Vaiont gorge, showing the pipe bridge carrying the water from Pieve di Cadore to Val Gallina, and immediately behind it, the excavation work in progress for the Vaiont dam

Some SADE Developments

We conclude our account of the developments in the Piave basin, making special reference to the Vaiont dam, which, it is believed, will be the highest arch dam in the world. The remainder of this article is devoted to a description of the Cellina scheme

PART FOUR

WE have deferred until this month the conclusion of our description of the Piave-Boite-Maè-Vaiont development, as we wished to draw special attention to one of the highlights of this exceedingly interesting scheme—the Vaiont dam. This dam, which was designed in the SADE Hydraulic Construction Department under the direction of Dr. Semenza and is now being built by G. Torno, Milan, will, it is believed, be the highest arch dam in the world.

The Vaiont, it will be remembered, flows into the Piave from the left just below Longarone and just above the confluence of the Maè on the right bank. In flowing through the rocky parapet forming the left flank of the Piave valley, the Vaiont has cut a gorge of truly gigantic proportions. In Fig. 33 we attempt to give some idea of the depth of this gorge, but the task is frankly beyond the powers of photography.

From a constructional standpoint the rock could hardly be improved upon. It is a dolomitic limestone, compact, and in regular strata which dip upstream. Boreholes in the base and sides of the dam site have confirmed the excellence of the rock. The sides of the gorge are nearly vertical; in fact a width of some ten metres at the bottom enlarges to about a hundred metres at a height of 210 m. and to less than two hundred metres at a height of 260 m.

As far as the gorge itself is concerned it would be perfectly feasible to build a dam 300 m. high, which would create a reservoir of no less than 340 million cu. m. in capacity, but the design problem has been how best to integrate the Vaiont reservoir into the development scheme as a whole.

As we outlined in our previous article, the general basis of the scheme is to connect the Piave, Boite and Vaiont reservoirs hydraulically so that they function automatically as seasonal storage. To conform to this, the maximum level of the Vaiont reservoir would have to be 679 m. (the Pieve di Cadore reservoir stands at 683·5 m.), requiring a dam only 204 m. high and impounding 60 million cu. m. At any higher level the reservoir would have to be hydraulically separated from the rest of the system and connected thereto by some system of regulating valves.

Nevertheless, the possibilities of increased storage were so attractive that the complication of introducing control equipment between this reservoir and the Piave-Soverzene pressure tunnel was considered to be worth while. A factor that made the advantage decisive was that the Vaiont is strategically placed to form a balancing reservoir between the north-eastern Piave development and the future development on the Cellina immediately to the east, the hydrological characteristics of which differ from those of the Piave.

The 300 m. dam, which we have mentioned as being feasible, would have corresponded to a maximum storage level of about 760 m., but practical limits between 725 and 730 m. were set by the village of Erto and by the S. Osvaldo saddle connecting the Vaiont valley with Val Cimoliana. Finally, a crest level of 725·5 m. was chosen, with maximum storage level of 722·50 m. impounding 150 million cu. m. of water. The new reservoir will thus constitute a multi-seasonal balancing storage astride the Piave and Cellina catchments.

Owing to the considerable storage capacity and the maximum water level of the Vaiont reservoir as definitively designed, the following scheme has been studied for a complete filling of the reservoir. The waters of the Boite, up to a maximum flow rate of 10 cu. m. per sec., will be taken off at an elevation of 852 m. near Vodo (where one of the reservoirs of the projected developments of the upper Boite is located, as described in Part Three of this series) and conveyed, through a tunnel 9,237 m. long, having a diameter of 2·55 m., to the existing Maè reservoir at Pontesei, where the maximum level reaches an elevation of 800 m.: the difference between the two elevations will be utilised by an underground station at Pontesei (capacity 8·5 MW, maximum and minimum head 66 m. and 40 m. respectively, average yearly production 25 GWh).

The Pontesei power station will be located near the Maè dam and operate under counter-pressure, discharging the water directly into the existing tunnel which connects the Maè reservoir to the Gardona power station. After passing through the Piave river crossing, the water will be conveyed directly to the Vaiont reservoir through a tunnel 3,073 m. long and 2·75 m. in diameter. This tunnel is parallel to the Pieve di Cadore-Soverzene tunnel, though independent of it, and permits the water to rise to a maximum elevation of 722·50 m. Owing to the diversion of the Boite via the Maè reservoir, Gardona station will have its average yearly production increased by approximately 40 GWh.

The difference at any given time between the instantaneous level in the Vaiont reservoir and the piezometric level in the Pieve di Cadore-Soverzene pressure tunnel will be utilised by an underground station at Colomber (capacity 9 MW, maximum head 60 m., maximum flow rate 22 cu. m. per sec., average yearly production 10 GWh). Thus the water of the

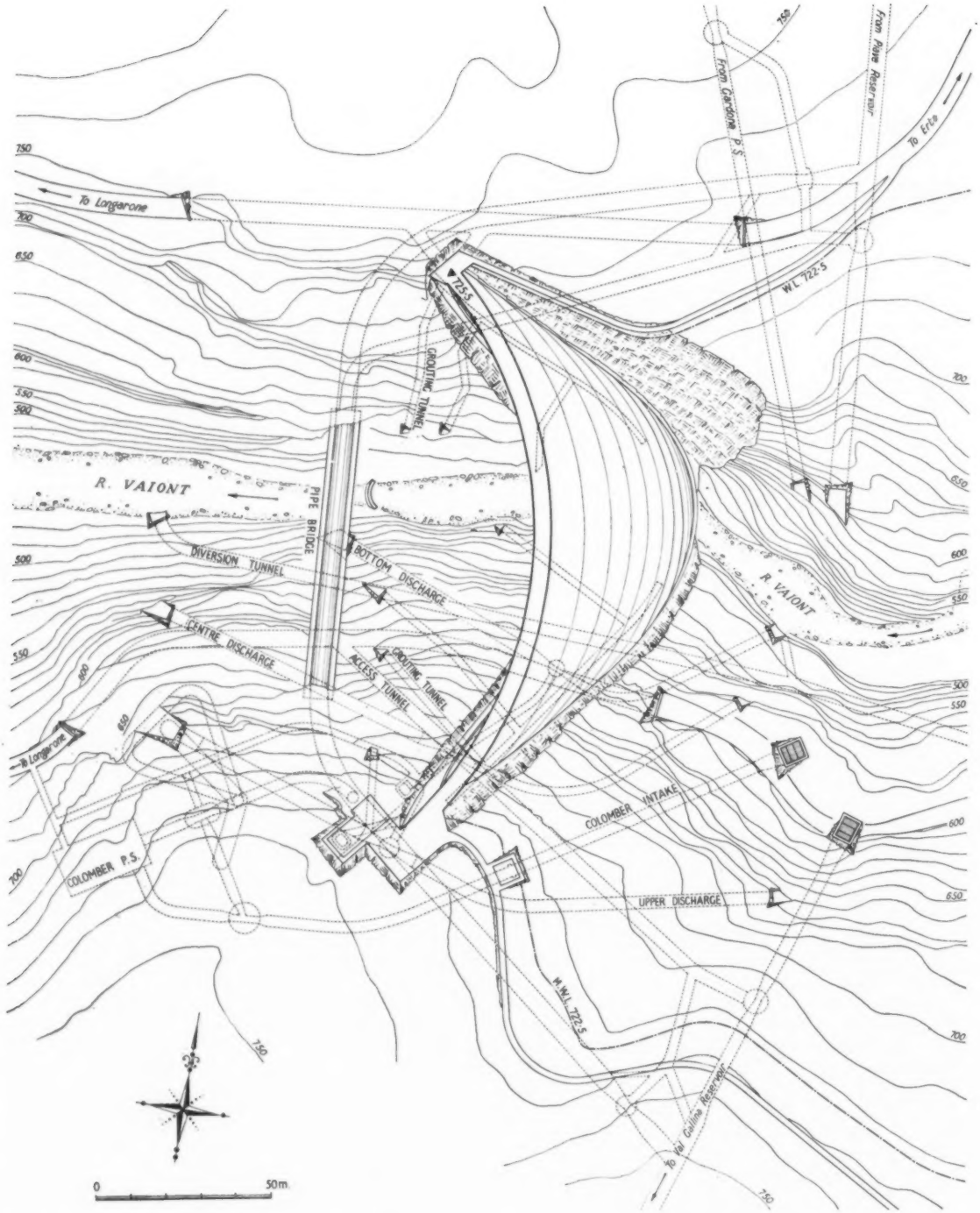


Fig. 34a. Plan of Vaiont dam showing the locations of the various tunnels and of Colomber underground power station

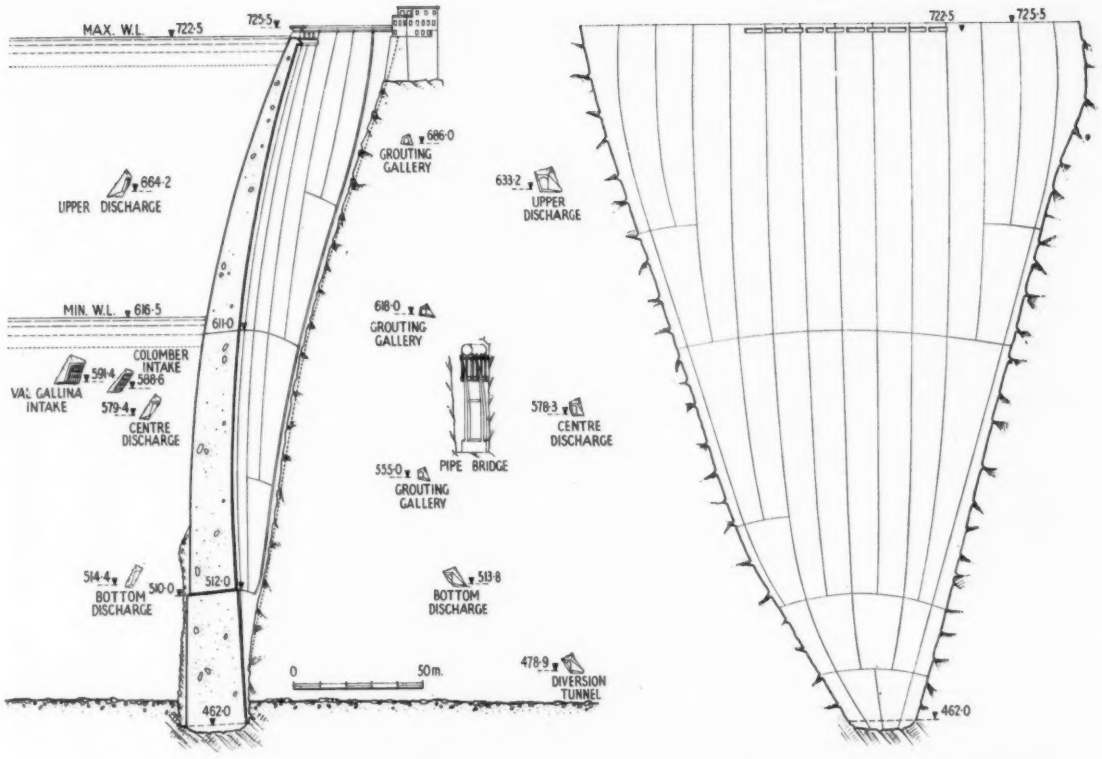


Fig. 34b. Elevations of Vaiont dam. At 265·5 m. high it will be the highest arch dam so far completed in the world

Vaiont reservoir will operate the Colomber station and be discharged under counter-pressure into the Pieve di Cadore-Soverzene tunnel, just downstream of the existing pipe bridge which spans the Vaiont gorge; thence it will be conveyed, together with the water coming from the reservoirs of Pieve di Cadore and Valle di Cadore, to the reservoir of Val Gallina and from there to Soverzene station. Obviously, when the level of the Vaiont reservoir sinks as low as the levels of the other upstream reservoirs, taking into account the head losses, the waters of the Vaiont can be conveyed to the tunnel directly.

The construction of the above scheme has just started.

The Vaiont dam site is just upstream of the pipe bridge (Fig. 33) carrying the water in the Piave-Soverzene pressure tunnel across the Vaiont gorge. Drawings of the dam are given in Figs. 34a and 34b. The dam will be 265·5 m. high, it will have a crest length of 190·5 m. and a chord of 168 m., it will be 21 m. thick at base, and will contain 360,000 cu. m. of concrete. It will be a dome-shaped structure with a peripheral joint.

In spite of the size of the dam it is not anticipated that there will be any exceptional difficulties in its construction. Various model studies have confirmed the design calculations, and in particular, have demonstrated the large proportion of the load carried by the rock walls, the lightness of the stresses in the lower part of the dam enabling a comparatively thin bottom plug to suffice, and the regularity of the load-

Fig. 35. A view of Val Gallina dam

Fig. 36. Interior of the completed Soverzene station

ing pattern, notwithstanding a marked asymmetry towards the top of the dam.

The contractors for the Vaiont dam are as follows: *Civil works*, Impresa Dott. Ing. Giuseppe Torno & C., Milano; *Grouting*, Impresa ICOS (Impresa Costruzione Opere Specializzate), Milano, and Impresa Consonda (Impermeabilizzazioni Cementazioni e Consolidamenti), Milano; *Road bridge across the gorge below the dam*, Impresa SACAIM (Soc.p.Az. Cementi Armati Ing. Mantelli, Venezia.

Val Gallina Reservoir

The function of Val Gallina reservoir as daily and weekly storage for Soverzene station, leading to striking economies in the construction of the tunnels and of the Soverzene surge tank, has already been explained. In fact, the savings have amply justified the cost of the dam. At the dam site the condition of the rock was doubtful and the profile very irregular, and the most exhaustive investigations were needed before a solution was found affording the requisite degree of confidence.

Val Gallina dam (Fig. 35) is a double-curvature arch with a pronounced overhang downstream. Symmetry has been obtained by blocking the bottom of the gorge with a plug, the arch itself being seated on a peripheral joint. Both upstream and downstream faces are steel reinforced. The dam is 92·37 m. high, has a crest length of 228 m., a top chord of 190 m., and a total concrete volume of 99,000 cu. m. The reservoir has a gross capacity of 6·4 million cu. m. and a full storage level of 677 m.

It will be recalled that one tunnel from Pieve di Cadore discharges to this reservoir, whereas two parallel tunnels leave it for Soverzene. A bypass tunnel has also been driven to connect the upstream and downstream tunnels, and this tunnel can deliver to either or both of the Soverzene tunnels. Thus, quite an elaborate system of gates has been needed for the control of the waters at this reservoir.

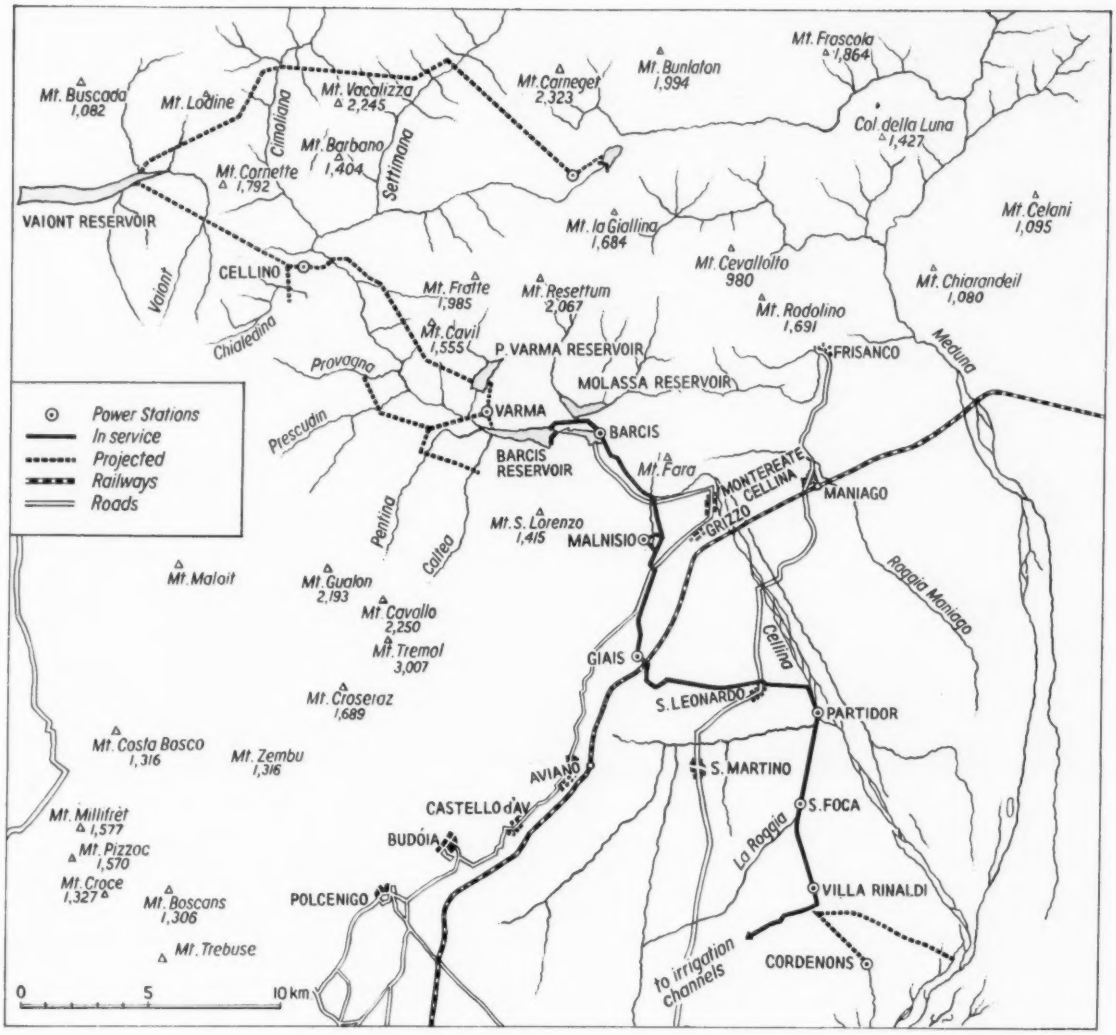


Fig. 37. Map of the developments on the Cellina

Soverzene Power Station

This interesting and handsome station has already been fully described in this journal, and we cannot do better than refer our readers to that account.* Since that article was published, however, the fourth 55 MW Riva/Marelli generating set has been installed, and we are able to reproduce in Fig. 36 a view (unfortunately not in colour) of the machine hall in its final state.

On this question of colour we should like to comment on the artistry of the decorative scheme throughout the station. The Italian's genius in the use of marble, tiling, mosaic, and fresco, not to mention the more humble decorator's paint, is to be seen to advantage in most of his power stations, and at Soverzene no pains have been spared to achieve a striking and beautiful result. The effect is most harmonious and its impact on the visitor is impressive.

* "Soverzene Generating Station." By Dr. Ing. Mario Mainardis. WATER POWER, March and April, 1952, pp. 95 and 124.

DEVELOPMENTS ON THE CELLINA

The Cellina is a right-bank tributary of the Meduna, itself a left-bank tributary of the Livenza, and its basin lies immediately to the east of that of the Piave. Its catchment area is 424 sq. km., and in view of its high specific run-off (49·1 litres per sec. per sq. km.) it forms an attractive proposition for hydro-electric development. Irrigational use of the river is also of great importance to relieve the aridity of the plain in the region of Pordenone, and the later developments have been planned and built by SADE in collaboration with Consorzio di Bonifica Cellina Meduna—the Board of Reclamation responsible for the district.

A map of the catchment showing the existing and project developments is given in Fig. 37. Plant now in operation has a total capacity of 35,700 kW and generates 230 million kWh in an average year, and when the developments now planned are completed, the installed capacity will be 200 MW and the average annual production 670 million kWh.

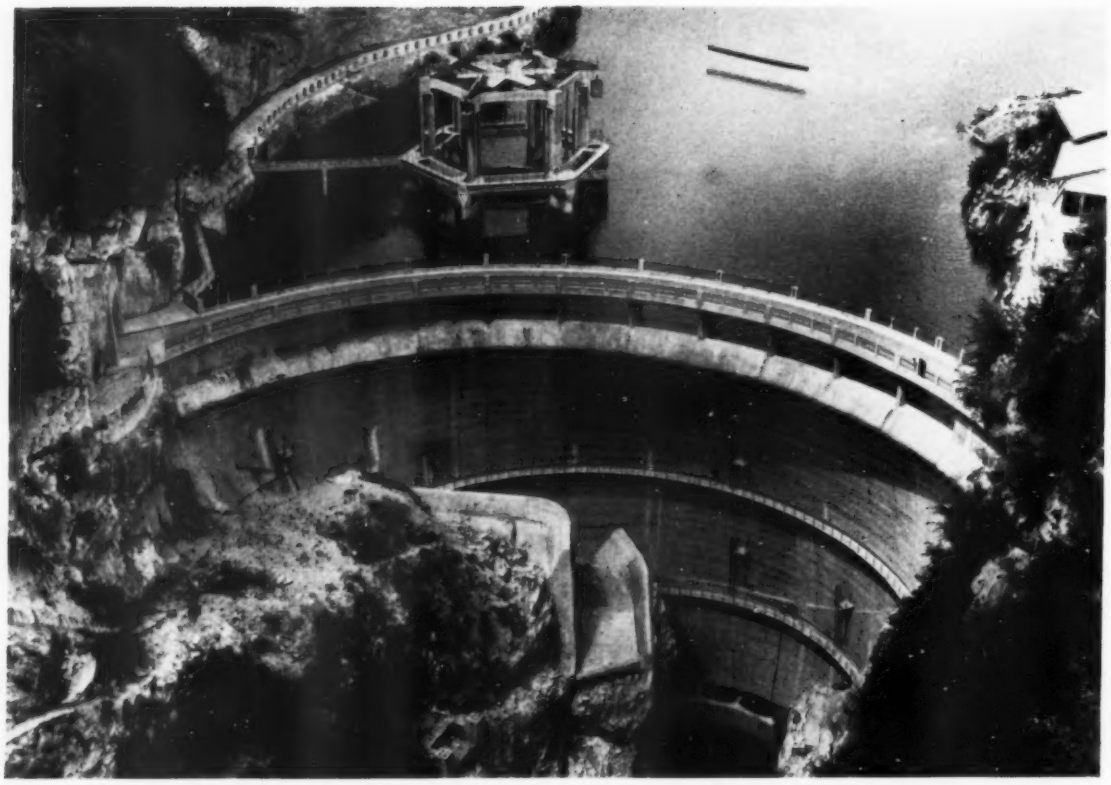


Fig. 38. Barcis dam, showing the controlled circular spillway in the background

The lower course of the river has been exploited for over 50 years by a group of three free-flow stations in series between the levels of 342·6 and 196·6 m. Malnisio station, the uppermost of the three, has been in operation since 1905, and contains four horizontal Francis sets with an aggregate capacity of 6,000 kW, working under a head of 54·5 m. Giais station, the next in order, entered into service in 1908, with two 4,000 kW Francis sets under a head of 56 m., and Partidor station completed the series in 1919, having two 1,250 kW horizontal Francis sets under a head of 26·5 m. The maximum discharge through each of these stations is 14 cu. m. per sec.

These rather old stations became the nucleus for a comprehensive development of the Cellina catchment. As a first stage a head reservoir has been constructed at Barcis, supplying a power station of the same name, and below Partidor station two stations—S. Foca and Villa Rinaldi—have been built in conjunction with an enlarged irrigation network. A scheme is projected for developing the upper catchment above Barcis, and a final power and irrigation station is planned below Villa Rinaldi.

Barcis Dam and Reservoir

Barcis reservoir forms the head storage for the lower Cellina development, and has been formed by impounding the river at Ponte Antoi gorge to give a useful capacity of 20 million cu. m. at a level of 402 m.

From a geological standpoint the site was a difficult one as it was in the region of a fractured zone of Eocene rock between the dolomite and the limestone. The site of the dam itself was completely in the limestone, but as the fissures were parallel to the river bed and extended downstream to the side of the dam site it was necessary to supplement the dam by a huge grout curtain, more than 200 m. long, to connect the limestone to an impervious bed of red sandstone. This curtain was formed by boreholes 60 m. deep and 5 cm. in diameter and spaced at an average distance of 2½ m. Sodium silicate was injected in advance to ensure fluidity, and was followed by a fine-grained grout having a water-cement ratio of 1 : 1 by weight and mixed with calcium bicarbonate for rapid hardening. An injection pressure of 25 atm. was used at the bottom of the holes, and was decreased as the surface was approached, grouting being carried out in 5 m. stages from the bottom upwards. Residual leaks were detected by dyes and injected locally.

This grout curtain actually cost more than the dam itself, and absorbed 100 tons of dry cement.

The faulted zone also called for the utmost care in constructing the road round the lake, as the dip of the strata was towards the lake.

Barcis dam, depicted in Fig. 38, is a dome-shaped structure with peripheral joint, but it displays the most recent tendency in SADE dam design in that an asymmetric structure (see Fig. 39) has been accepted to suit the profile of the site. This contrasts with earlier practice as exemplified, say, at Val Gallina, where considerable excavation was undertaken in the left flank of the site to secure a structure that was symmetrical within its peripheral joint. The dam is

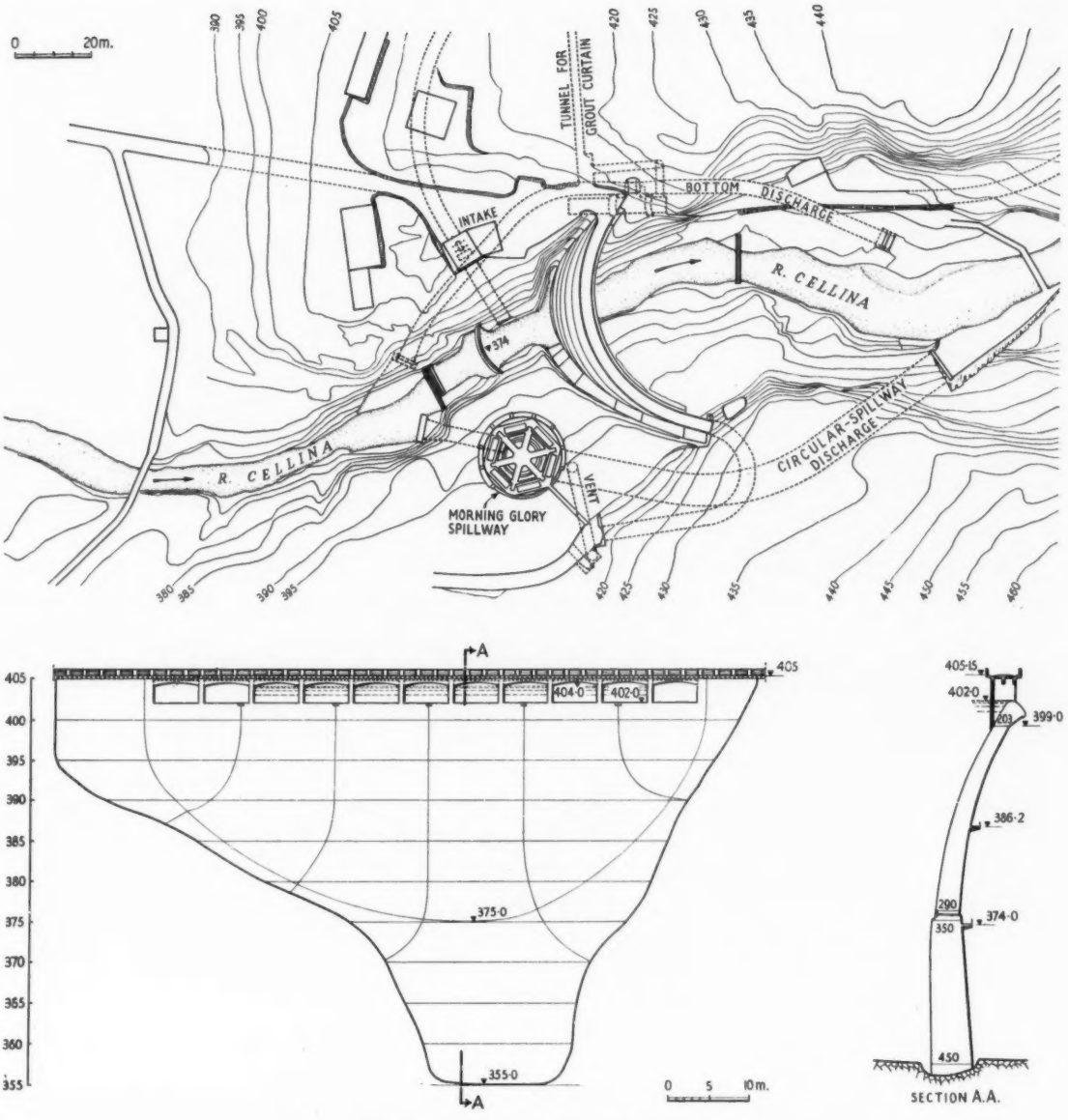


Fig. 39. Plan and elevation of Barcis dam

50·15 m. high, has a crest length of 83·7 and a top chord of 62·85 m. It contains 9,000 cu. m. of concrete. It was built by Impresa F.lli Monti, Auronzo, the grouting being performed by ICOS.

The Cellina catchment is impervious, giving a rapid run-off, so that the river is characterised by heavy floods, and it was considered necessary to design the Barcis works for a calculated flood of 1,400 cu. m. per sec. For this purpose the greater part of the dam crest is occupied by eight free spillway openings, each 11 m. wide and allowing for a flood level 2 m. above the sill. As the dam is comparatively short, however, this provision is not nearly sufficient to handle the design flood, so the bulk of the flow is taken by a glory-hole spillway, seen in the background in Fig. 38, capable of discharging 970 cu. m. per sec. In addition, there is a bottom outlet on the left bank with a sill level of 360·18 m.

The circular gate of the glory-hole spillway is operated by oil pressure from a pump which can be driven either by an electric motor or by a small water turbine. This turbine starts automatically when the water rises above maximum retention level.

The power intake from Barcis reservoir is located in the left bank immediately above the dam, and communicates with a concrete-lined pressure tunnel, 3·9 m. in diameter and about 2 km. long, leading to Barcis station. This tunnel runs parallel to the Cellina valley at a minimum distance from the valley wall of 40 m., and over one length of about 100 m. a crack in the rock cover called for the concrete lining to be reinforced. During its course the tunnel crosses the

gorge of a minor tributary, the Molassa, by means of a self-supporting pipe bridge of 18 m. span.

Near the power station the supply tunnel terminates in a restricted-orifice overflow-type surge shaft 8 m. in diameter with an annular expansion chamber at the bottom. A penstock 2·5 m. in diameter and 19 m. long runs from the base of the surge tank to the turbine, provision being made for a second penstock when the station is completed. These works have been constructed for a total flow of 44 cu. m. per sec. and a maximum head of 58 m. at the turbines.

Barcis Power Station

Barcis power station, depicted in Figs. 40 and 41, is of the semi-underground type in the sense that it is located in a recess cut in the face of the rock. It is situated directly opposite to the channel intake supplying the old Malnisio station, so that the Barcis tailwaters merely cross the bed of the river. Spanning the river immediately downstream of these two structures is a weir containing four scour sluices.

Space is available in the station for two generating sets, but at present only one is installed. This is a 10 MVA vertical unit consisting of an Escher Wyss Kaplan turbine coupled to a Marelli generator. When installed the turbine was believed to be the highest-head Kaplan machine in the world, and is designed to operate on an unusually wide range of head—from 58 to 28·5 m. To secure the utmost efficiency over such a wide range the governor cabinet is provided with an eight-position hand control which alters the setting of the combinator gear to suit the actual head on the turbine. To prevent cavitation the turbine is placed 3·9 m. below the minimum tailrace level, but as the tailrace level fluctuates as much as 7·9 m. it was necessary, in order to protect the generator from flooding, to provide a clear length of shaft of about 11 m. between the turbine cover and the feet of the generator. The overall length of this shaft from turbine runner to exciter is about 20 m. To control this length of shaft a turbine guide bearing, two intermediate bearings and two generator bearings have been provided, in addition to the thrust bearing, which in this in-

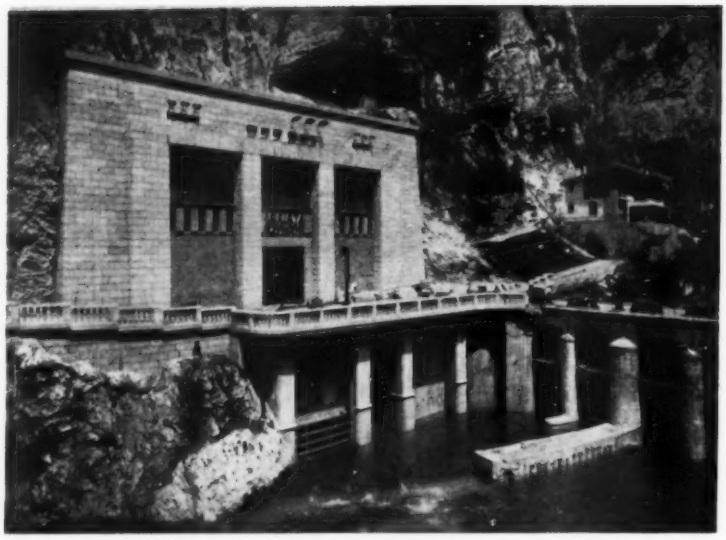


Fig. 40. Exterior of Barcis power station

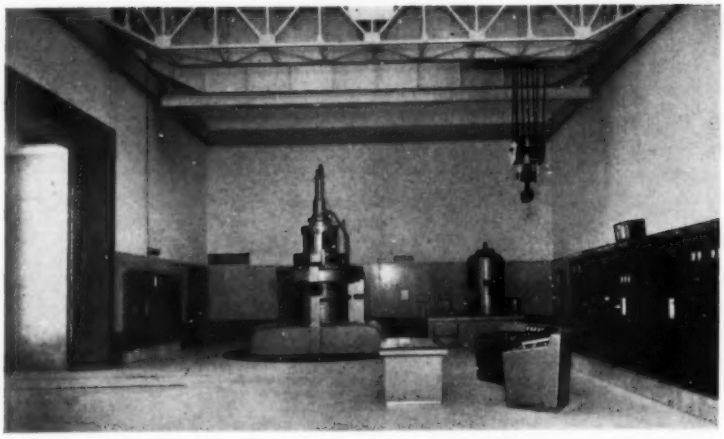


Fig. 41. Interior of Barcis power station

Fig. 42. Interior of S. Foca power station

Fig. 43. Outlet canal from S. Foca station (seen in the distance) and intake for Villa Rinaldi station and the Cellina irrigation system

stance has been located on the top of the generator.

Needle bearings are used on the guide-vane links to reduce the adjusting forces necessary. To avoid heavy stresses on the operating mechanism under the high head in the event of an emergency closure, a quick-acting rotary valve is placed immediately ahead of the turbine and is tripped to close automatically in the event of an overspeed. Simultaneously air is injected to prevent the runner lifting under vacuum formation.†

The Marelli generator supplies 8·5 MW at 0·85 power factor at 6 kV, and the power is stepped up to 60 kV by a 10 MVA Marelli transformer housed in a cubicle to one side of the entrance hall. The high-voltage leads are brought out through bushings in the façade of the station to an outdoor switchyard where the supply is controlled by a Galileo minimum-oil type 80 kV 600 A circuit breaker having rupturing capacity of 1,500 MVA. The switchyard is arranged on the single-busbar system, and a second similar circuit breaker controls a 60 kV line to Giais.

In addition to the circuit breakers, all the isolators, instrument transformers and similar equipment in the switchyard, as well as all the switchgear instruments and protective relays within the station, were supplied by Galileo. Within the station there is no control room as such. As will be seen from Fig. 41, the five-panel control desk is placed in the machine hall itself, facing a seven-panel vertical board carrying the instrument and protective equipment. There is also a seven-panel board for the 10 kV inter-station service lines and for station auxiliaries.

S. Foca and Villa Rinaldi Stations

S. Foca and Villa Rinaldi stations, which successively use the Partidor tailwaters, are virtually identical, and the following description of S. Foca station applies almost equally well to Villa Rinaldi. Both have been designed as power units in what is essentially an irrigation network. S. Foca operates between the levels of 191·4 and 136·7 m. and has a working head of 55·3 m.; for Villa Rinaldi the levels are 135·9 and 83·3 m. and the working head 50·6 m.

From Partidor station a short free-flow open channel of parabolic section conveys the tailwaters to a forebay whence a penstock leads down to S. Foca station. Provision is made to install two further penstocks when the station is completed.

The penstocks for the two stations are both of particular interest as they are constructed in pre-stressed-concrete sections 5 m. long and 2·65 m. in internal diameter, following two quite different methods.

The S. Foca penstock, about 2,700 m. long and 10 cm. thick, is constructed by pouring normal concrete between vertical steel shutterings, containing only a light reinforcing steel net. After pouring, the pipe is treated in a vacuum cell for 1·30 hours with a 30 cm. Hg vacuum in order to extract the excess water (ultimate water-cement ratio about 0·34). Then the structure is cured for 6 hours under hot steam (70°C.) and is finally transferred to a yard. Following such treatment, 24 hours after pouring, the pipe has an approximate resistance of 300 kg per sq. cm.

The pipe is then prestressed by means of a special wire-drawing machine with high-tensile steel wire (4—5 mm. in diameter, ultimate stress 140—150 kg per sq. mm.) and with a pitch of turn variable according to the internal hydraulic head. In such a way, and by reducing the wire section through a die, an ultimate stress on the steel of 100 kg. per sq. mm. is obtained. During winding, the machine is apt to prestress the pipe longitudinally as well, and the elastic

† An account of the acceptance tests on this turbine is given in *Escher Wyss News*, No. 1, 1956, p. 5. An abstract of this article was published in WATER POWER, May 1956, p. 200.

Fig. 44. Prestressed conduit being laid to Villa Rinaldi station

deflections are controlled by means of electro-acoustic gauges having an approximation of half a micron. Finally the spiral on the pipe is protected by guniting the surface with cement mortar.

The pipes are laid on a continuous saddle and are also prestressed longitudinally three by three by means of 33 high-tensile steel wires, obtaining in the concrete an ultimate stress of about 5 kg per sq. cm.

The prestressing method adopted for the Villa Rinaldi station pipes is completely different. This penstock, about 2,350 m. long, consists in the co-operation of an outer concrete sheath, 3 cm. thick, containing a high-tensile steel spiral (5—6 mm. in diameter) and of an internal concrete cylinder 8 cm. thick, put in a state of prestress. The prestressing of the outer steel spiral is made to act directly on the internal concrete cylinder by means of hydraulic pressure within a series of rubber-impregnated fabric sleeves, the pressure being maintained constant until the concrete has completely hardened. The hardening is accelerated by circulating hot water (70°C.), through a system of pipes and is achieved during a period of 14–18 hours. The ultimate stress in the steel wires, is 105 kg. per sq. mm. and the average elasto-plastic flow in the cement, after this operation, is about 10%.

For both penstocks one pipe in every ten is tested at a pressure 30% higher than the operating head, and, for experiment, the pressure has been raised to 3 times the working value with excellent results and without having been able to break the pipe.

The joints (one every 15 m. for S. Foca, and one every 5 m. for Villa Rinaldi) have been made by internally sealing a layer of plastic material into their seats by means of a thin wrapper of ferro-cemento.

About 30 m. upstream of S. Foca station is a reinforced-concrete surge tank (Fig. 43) 50 m. high, and thence a buried pipe connection partly in reinforced concrete and partly in steel, leads to the station.

S. Foca station, an interior view of which is given in Fig. 42, is at present equipped with a single 5,500 kVA vertical Francis set consisting of an Escher Wyss turbine driving a Marelli generator. To meet irrigational requirements the turbine is fitted with a pressure relief valve, rigidly connected to the turbine wicket gates; the valve is automatically opened in the event of a reduction of load on the turbine, so ensuring that a constant flow is maintained in the tailrace.

A Marelli transformer steps up the power from 6 to 60 kV, and, as is the case throughout the later SADE stations, all switchgear, instruments and protective gear are of Galileo manufacture.

Eventually it is hoped to install a second generating set of twice the capacity of that now in service. The station discharge will then be increased from 12 to 36 cu. m. per sec.

The S. Foca tailrace, like that of Partidor, is of parabolic section in reinforced concrete, and irrigation channels are taken off from either side before it terminates in a forebay (Fig. 43) serving the Villa Rinaldi penstocks.

Ultimately a third and final station will be built at Cordenons, the discharge from which will go entirely to irrigation.

Future Upper Cellina Scheme

We have now described the existing works on the Cellina and can conclude this section with a brief sketch of the plans to develop the upper catchment.

As we explained earlier in this article, the Vaiont reservoir is intended to form a balancing multi-seasonal storage between the Piave and the Cellina developments, and it therefore forms an integral part of the projected system. The intake of this reservoir will be supplemented by intercepting the headwaters of the upper Cellina, the Settimana and the Cimoliana Rivers and diverting them up to a maximum discharge of 15 cu. m. per sec. to the Vaiont reservoir. An offtake from this reservoir will lead to a station at Cellino, picking up the flow from the Chialedina on the way. This station will operate on a maximum head of 227 m. and will develop 6·5 MW with a discharge of 35 cu. m. per sec.

The Cellino tailwaters will be conducted to a reservoir at Ponte Varma on the River Varma supplying a station of the same name which will also be fed by four right-bank tributaries of the Cellina—the Provagna, Prescudin, Pentina and Caltea. Varma station will develop 9,800 kW at a head of 85 m., and will discharge 40 cu. m. per sec. to Barcis reservoir.

These developments will improve the regulation of the river to such an extent that it will be possible to complete the installations in Barcis, S. Foca and Villa Rinaldi and also to increase the output from the three old stations, Malnisio, Giais and Partidor.

(To be continued)

Busbar Clamps. List F5 received from Ferranti Limited illustrates and gives specifications for their busbar clamps; these are non-magnetic made from high-strength ductile material, and are supplied ready for assembly.

Power Penstocks

Analysis of various penstock designs and construction methods

by P. J. Bier, Consulting Engineer, formerly in charge of penstock design of the U.S. Bureau of Reclamation

PART ONE

THE principal purpose of this article is to discuss in some detail the latest design and construction methods as applied particularly to penstocks supplying water under pressure to hydraulic turbines. Penstocks should be constructed to fulfil both the requirements of safety and economy. Rules for the construction of pressure vessels are incorporated in the ASME code for fusion-welded, unfired pressure vessels. Although this code was prepared for self-contained pressure vessels, the rules contained therein are with some exceptions also applicable to pressure pipe. They have been used for such purposes for many years, as there is no specific code available for large steel pipes.

A comparative analysis has been made of four different penstock designs, classified as A, B, C and D. All four designs are based on identical profiles and plan alignments, using 78 in. and 72 in. diameter pipes, which are welded into suitable shipping or erection lengths in the fabricating plant. Depending on the shell thickness, the pipe sections are joined at the site either by welding or riveting. These field girth joints may also consist of Dresser couplings for any pipe shell thickness.

The four penstock designs are described as follow:

A. This design consists of pipe sections shop-welded in 40 ft. erection lengths and provided with double ring girder supports, rockers, and concrete support piers spaced at 60 ft. centres. The field girth joints are welded for plate thicknesses 1 in. or less and riveted for thicknesses exceeding 1 in. A carbon steel A285 gr. C is used throughout this design. The pipe shell thickness is proportioned primarily for the internal pressure except at the supports.

B. This design is also based on 40 ft. shop-fabricated sections which, however, are joined with Dresser couplings in the field. No ring girders or stiffener rings are used, but the pipe is supported on saddle-type concrete piers spaced at 40 ft. centres. The pipe shell thickness is proportioned primarily for the internal pressure. Two types of steels, A285 gr. C and A212 gr. B, are used in this design. All bends except that at the downstream end are formed by means of 40 ft. pipe sections laid with 2° deflection angles in each coupling.

C. This design is identical in every respect with B using two types of steels with Dresser couplings as field girth joints, excepting that the shell thickness is computed for a combination of stresses due to the internal pressure and the circumferential bending moment at the saddle supports.

D. This design is also identical with B, using two types of steels with Dresser couplings, except that single ring girders supported on slide plates are provided at the piers. A pier spacing of 40 ft. is used and the piers are of similar shape to those for design A. The shell thickness is proportioned primarily for the internal pressure.

The analysis is based on the provisions of the 1956 edition of the ASME code. As indicated above, two types of steel plates are used for the coupled designs, both of firebox quality—a carbon steel of ASTM Designation A285, gr. C, and a high-strength carbon-silicon steel of ASTM Designation A212, gr. B. The A285 is a semi-killed steel and A212 is a fully-killed steel, and both are cross-rolled and subjected to homogeneity tests to assure uniform quality. The code limits the thickness of steel to be welded without subsequent stress relieving for both types to 1 in. or less for the affected 72 in. diameter portion of the penstock, in accordance with the formula of $\frac{D+50}{120}$ The upstream 78 in. diameter portion of the penstock which is under a lower head and has a thinner shell is not affected by this rule. This code provision also limits the thickness of the pipe shell to be welded for the field girth joints to 1 in. or less, as local stress relieving of the joint after welding in the field is not feasible. Consequently riveting or some other means of plate joining must be resorted to for large steel pipes exceeding 1 in. in shell thickness.

LIST OF SYMBOLS

D = inside diameter of pipe (in.); r = inside radius of pipe (in.); p = internal pressure in lb. per sq. in.; s = design stress in lb. per sq. in.; t = plate thickness (in.); e = efficiency of joint; w = unit weight of water 62·4 lb. per cu. ft.; A = cross sectional area of the pipe (sq. ft.); H = max. head including water hammer (ft.); V = velocity of flow in ft. per sec.; g = acceleration due to gravity (32·16); Q = flow in cusecs.

Neither riveting nor manual welding of field girth joints is a very desirable or economical means of joining pipe sections. Manual welding of A212 steels in the field, where controls are lacking, is especially undesirable and not favoured by welding men. The principal objections to riveting are that riveting has practically become a lost art in pipe fabrication and it is difficult to find expert riveters at present. On the other hand, rivet heads projecting on the inside of the pipe, unless countersunk and ground flush, tend to obstruct the flow of water and reduce both the pressure head in a penstock and its carrying capacity. It is very difficult to produce a fully watertight joint by riveting unless extensive caulking or seal-welding is used, which increases the cost and slows down production. Field welding during erection is likewise a problem in many installations, especially if it has to be performed in such cramped quarters as trenches or tunnels, or under adverse weather conditions. Such difficulties slow down the work and affect the quality of the welds. As penstock installations are usually

located in remote areas, far from population centres, it is necessary to import qualified welders from distant places, often for only a comparatively small amount of work, and this also increases the final cost.

The ideal pipe line would be one with all longitudinal and circumferential or girth joints automatically welded, ground flush with the interior and covered with a smooth spun enamel coating to provide efficient flow conditions. As this at present is not within the realm of feasibility, a type of construction should be used which most nearly fulfills these conditions. A self-centring mechanical sleeve-type coupling of proper design and fabricated within close tolerances to ensure watertightness is believed to be the best solution of the problem. Such a coupling, if properly installed, is flexible, permitting slight deflections without leakage between sections, is comparatively easy to erect and can readily be maintained in watertight condition during service. In addition, the couplings absorb the movements due to temperature changes in the line and so obviate the necessity for the usual expansion joints between anchors. Intermediate anchors can be eliminated as each coupled pipe section is anchored at one end to the support pier. Seeing that the line deflections at bend points can be taken up generally by the permissible deflections of a number of couplings, the usual fabricated bends and bend anchors can be dispensed with. These couplings—Dresser couplings—have been applied to hundreds of miles of pipelines and have given satisfactory service under all conditions of installation, exposed and buried. In more recent years the use of these couplings has been extended to large penstocks.

As a basis of the analysis, a typical penstock has been selected of identical profile and plan for all four types considered. Each penstock consists of four tangent lengths on a profile running downhill from the intake to the turbine with slopes varying from $7\frac{1}{4}°$ to

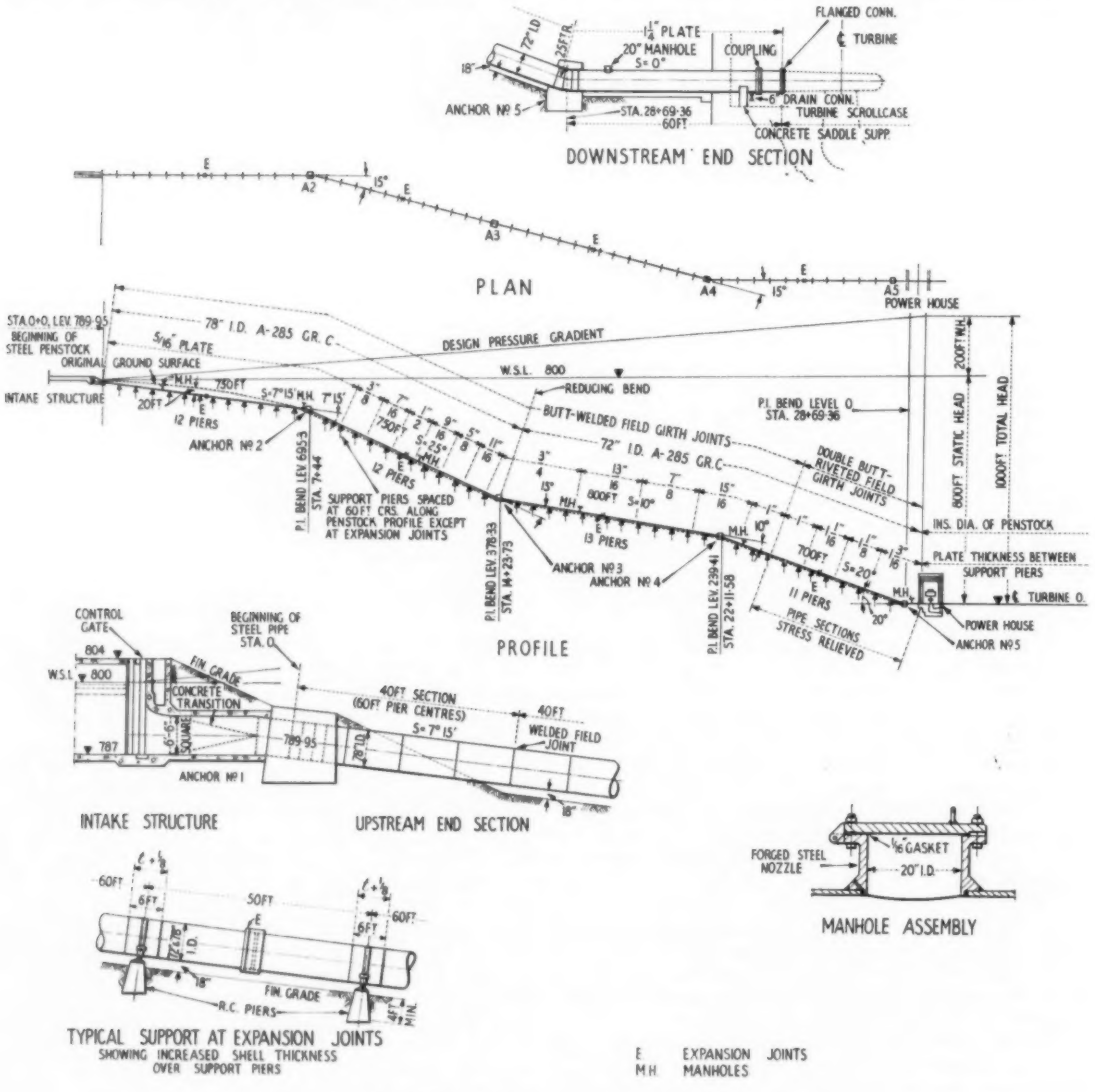


Fig. 1. Welded and riveted penstock—plan and profile

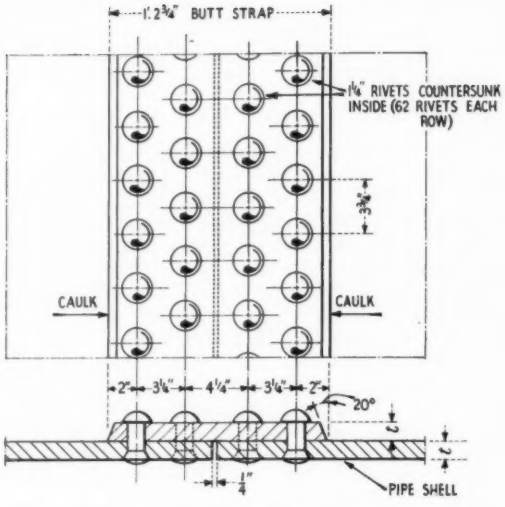


Fig. 2. Riveted field girth joint

25°. As already indicated above, each penstock is designed with two diameters, the upper portion being 78 in. diameter and the lower half reduced to 72 in. diameter with the object of economising in the weight of steel used for this high head portion which requires a greater thickness of shell. The economical diameter is determined by a special study in which the cost of the penstock is balanced against the value of the power lost in pipe friction. The diameter which results in the lowest annual cost, including these values plus the fixed charges, for each size is the most economical one to use. The penstock carries an average flow of 425 cusecs to a vertical-shaft single-runner reaction turbine rated at 30,000 h.p. The static head varies from about 13 ft. at the intake to 800 ft. at the turbine, which, with a maximum waterhammer head of 200 ft. based on a gate closure of 10 sec., gives a total head varying from about 13 ft. to 1,000 ft., which is represented by the design pressure gradient plotted on the profile of each penstock. This total head was used in the calculation of the pipe shell thickness along the penstock profile.

A concrete-lined canal supplies the water to the intake structure of the penstock, and the flow into the penstock is controlled by a fixed wheel gate. The intake structure is provided with a trash rack to prevent floating matter from entering the penstock, and with a siphon spillway to waste surplus water when the wicket gates of the turbine suddenly close due to a power failure. The upstream end of the penstock is provided with three seal rings encased in an anchor to prevent seepage. This anchor is connected to the intake structure by means of a rubber seal encased in both the concrete transition and the anchor.

At the downstream end the penstock is flanged to facilitate its connection to the turbine scroll case. A Dresser coupling is installed upstream from this flanged joint to permit adjustments in the alignment of the adjoining flanges.

Manhole nozzles with hinged covers are provided on top of the penstock to make it accessible for inspection and maintenance, and there is a drain at the downstream end for unwatering purposes.

Each design has been developed in sufficient detail to permit the preparation of quantity and cost estimates for comparison. Included in the estimates are the ring girders, rockers, concrete piers and anchors for Design A; the couplings, piers and anchors for Designs B and C; and the ring girder supports, couplings, piers and anchors for Design D. All earth work, including excavation and backfill, is also included in each estimate. Applicable unit prices from recent bids have been used as far as possible in estimating the cost of fabrication and installation for the steel pipe, and the cost of concrete and earth work.

A more detailed description is given in the following pages for each design, including the assumptions, loads and stresses used in each case. Also references are given to the sources of the information used and the computations on which the designs are based. Design and construction methods and practices which have been proved in service were used for each type, keeping in mind that an unsafe penstock may endanger an entire power project. This has been demonstrated in a number of penstock failures which caused loss of life and property.

Design "A"

Fig. 1 shows the general arrangement of Design A with profile, plan and essential details. This penstock is of all-welded construction, with the exception of the field girth joints for pipe sections exceeding 1 in. in thickness, which are riveted. The penstock is shop-fabricated in 40 ft. erection lengths and transported to the site for field installation. Shop-welding for straight pipe sections is performed on automatic machines using a one or two-pass submerged arc weld depending on plate thickness or equipment available. Curved joints such as used in bends and branches are manually welded. The field girth joints for shell thicknesses up to and including 1 in. thickness are also manually welded. In manual welding single-welded butt joints are generally used for plate thicknesses up to and including 1 in. and double-welded butt joints for plates exceeding 1 in. The plate edges for the thickness range used in this penstock are prepared for 60° V joints. All field girth joints for plate thicknesses over 1 in. thickness are riveted, using a double-riveted butt joint with outside butt-strap. The rivets are of $1\frac{1}{4}$ in. nominal diameter driven in $1\frac{5}{16}$ inch drilled and reamed holes using countersunk heads on the inside as shown in Fig. 2. The rivets on one side of the joint are driven in the shop and on the other side in the field.

A firebox quality carbon steel of ASTM Designation A285 grade C is used throughout this penstock and all butt-welded joints in the pipe shell and between the stiffener ring segments are fully radiographed. This increases the joint efficiency from the basic as-welded efficiency of 80 per cent. to 90 per cent., permitting the use of a net design stress of $13{\cdot}750 \times 0{\cdot}90 = 12{,}375$ lb. per sq. in. in computing the pipe shell thicknesses for sections not stress relieved. As the code calls for stress relieving for pipe sections made from carbon steel exceeding $1\frac{1}{4}$ in. in thickness or for thinner plates exceeding $\frac{(D+50)}{120}$ in thickness, the pipe sections for the affected 72 in. diameter exceeding 1 in. in thickness are stress relieved. For this stress-relieved portion, which includes

shell thicknesses from 1 to 1 3/16 in., the code permits a joint efficiency of 95 per cent. which gives a design stress of 13,750 × 0·95 = 13,062 lb. per sq. in. for proportioning the shell thicknesses. The plate thicknesses are computed from the membrane formula $t=\frac{rp}{se}$, with a minimum thickness of 5/16 in. for the upstream, low-head section to satisfy the requirements of rigidity during fabrication and handling. The shell thicknesses used vary from 5/16 in. at the upstream end to 1 3/16 in. at the downstream end, as shown on the profile in Fig. 1.

Bends between the tangent lengths are fixed in place by means of reinforced concrete anchors, so proportioned to meet the combination of forces acting on them. Sleeve-type expansion joints of conventional design are used between anchors; they act as construction joints, taking up discrepancies in pipe lengths during installation and absorbing expansion and contraction movements caused by temperature changes. Double ring girders are used at 60 ft. centres to maintain the circularity of the pipe and transmit the load by means of rockers to the concrete support piers. A pipe of this type, while circular, forms an efficient beam with a moment of resistance approximately equal to $r^2\pi t$. A 60 ft. pier spacing is used, this being considered to be about the most economical for the uniform ground profiles assumed in this analysis, for which only short support piers of low cost are required.

All bends are designed with long radii using small-increment deflections to reduce hydraulic losses to a minimum—an important feature in penstocks designed to conserve the pressure head. At the bends where line deflections are in two planes, combined bends are used and at the point of change in diameter from 78 in. to 72 in. a reducing bend is used.

The shell thickness at each ring girder support is increased by 1/8 in. over the adjoining shell to take care of secondary bending stresses. These are due to the restraint of the rigid ring girder to the expansion of the pipe shell when under internal pressure which, with full restraint, may reach a value of $1{\cdot}82\frac{pr}{t}$.

Expansion Joints. Fig. 3 shows a typical sleeve-type expansion joint as used for the 72 in. diameter penstock. It consists of inner and outer sleeves, joint packing, also packing gland, packing stop and gland bolts. Push-off bolts are provided for use when necessary to remove the packing gland from the stuffing box for repacking. Joint packing consists of 6 rings of braided, long fibre, lubricated flax as furnished by the Garlock Packing Company. Outside surfaces of the inner sleeves are nickel-clad to prevent corrosion and assure free movement in the expansion joint during temperature changes. The packing recess and the leg of the packing gland are machined to provide close tolerances between the moving parts. To facilitate the machining operation and provide dimensional stability after machining the expansion joint sleeves and packing glands should be stress relieved prior to machining. The length of the expansion joint is determined to accommodate a temperature variation of about 150° F. and to absorb any discrepancies in pipe length. A discrepancy of 1/8 in., plus or minus, may be assumed for each laying length in the distance between anchors for which the expansion joint is designed. Inner sleeves are proportioned for the end pressure applied on the packing by means of the gland and bolts to effect watertightness. This bolt load applied at a safe working stress produces an external compressive stress on the inner sleeve which must be within permissible limits to prevent distortion. The stresses in the packing gland are computed from the moment developed by the bolt force acting on the gland flange using yield point values in the bolts. To

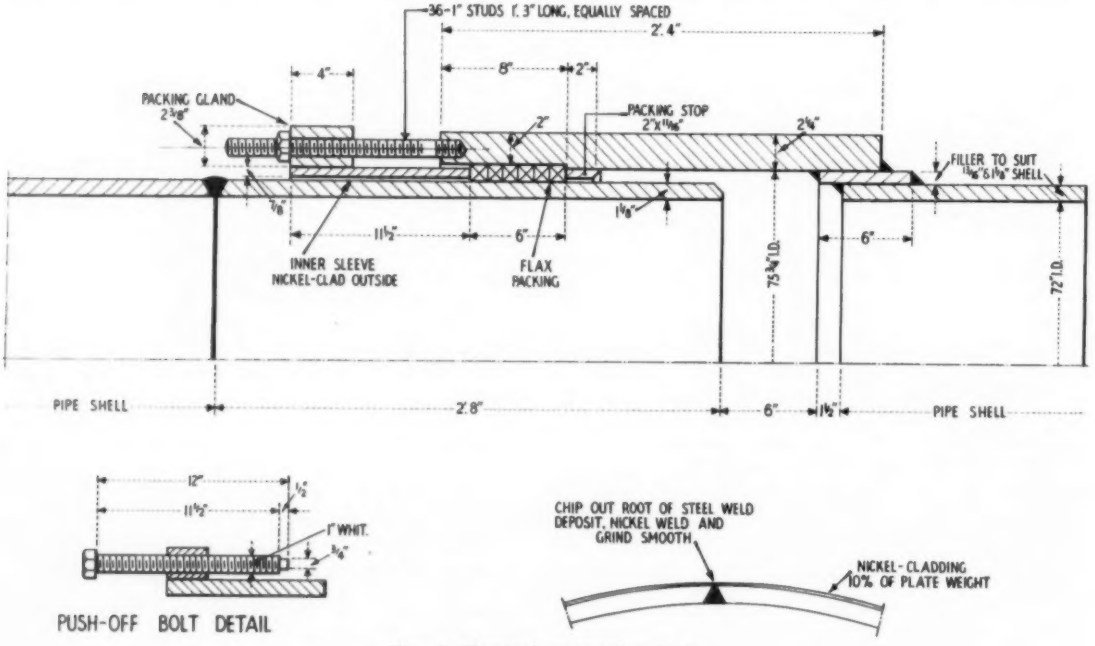


Fig. 3. Typical expansion joint

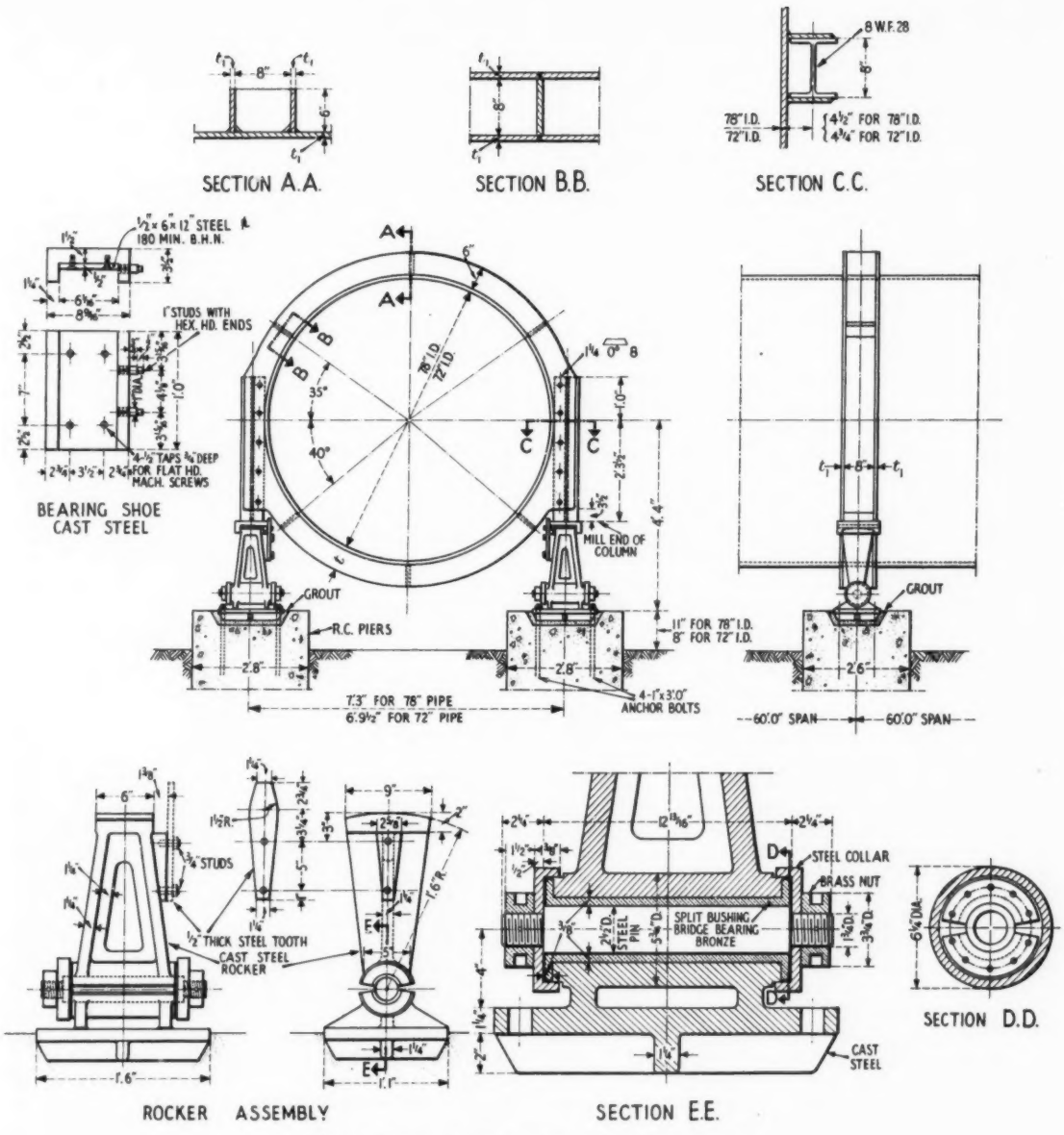


Fig. 4. Typical ring girder and rocker support

assure a watertight expansion joint, it is important that all sliding parts are fabricated within close tolerances and that a sufficient number of packing rings are used. No rational method of determining the required packing depth has been developed to date. It is evident, however, that the depth of the packing and the number of packing rings to be used is dependent on the pressure for which the expansion joint is designed. In 26 successful installations analysed, with static heads ranging up to 1,000 ft., it was found that from 3 to 9 rings of packing were used providing depth from 1½ to 11¼ in. based on ½ to 1¼ in. square packing. Although the greater packing depths were generally used for the higher head installations, no consistent relationship was evident between head and packing depth. It may be safe to assume, however, that a packing depth of less than 1½ in. should not be used for heads up to 100 ft. and that at least ½ in. should be added for each additional 100 ft. head. The packing used generally provides years of satisfactory service. In water lines not continually filled, the packing has a tendency to dry out when the pipe is empty and subjected to sufficiently high temperatures to cause the packing lubricant to run out. Experience shows that repacking in such installations is necessary about every 7 years.

Ring Girders and Rockers. Exposed self-supporting steel pipes are generally provided with stiffener rings or ring girders at the points of support to prevent deformations in the pipe shell. Fig. 4 shows a

typical ring girder support with rockers for the 78 in. and 72 in. diameter penstocks. Each ring girder consists of two rings of steel plate 6 in. high and with various thicknesses depending on the head at the supports. The two rings are welded to the outside of the pipe shell using continuous fillet welds and are tied into a unit with a number of diaphragm plates welded between them. At the horizontal centre the rings are widened to accommodate the support columns, these consisting of H-beams to which they are connected by fillet welds and plug welds. The support columns are attached to the ring girders eccentrically with respect to the centroid of the ring girder to reduce the bending moment on it. Steel bearing shoes are welded to the milled ends of the two columns to transmit the pipe and water load to cast steel rockers which, in turn, transmit the load to the concrete support piers. The purpose of the rockers is to reduce the longitudinal frictional force acting on the piers due to the expansion and contraction of the penstock. With the reduced frictional force, piers of smaller size can be used than is possible with supports based on slide friction. Heavy steel pins in bronze bushings provided with lubrite inserts transmit the pipe and water load from the rockers to the base castings, which are grouted into the tops of piers during installation. Before grouting these castings in place, adjustments are made in the final setting of the rockers in accordance with the prevailing temperature. Such adjustments are necessary to ensure that the rockers provide effective support during the entire range of temperature with the pipe filled and empty. A setting diagram is usually provided to the installation contractor for that purpose.

The ring girders used in the design are of sufficient size to act as effective stiffeners and maintain the circularity of the pipe shell at the points of support. Theory and experience show that a pipe line with adequate ring girders at the supports does not require intermediate stiffeners. As the restraining action of the semi-rigid ring girder will produce a narrow zone of secondary bending stress at the rim it was necessary to increase the pipe shell thickness for about 3 feet each side of the girder to reduce the stress to permissible values. This portion of the shell must be proportioned for a combination of the above secondary stress and the maximum longitudinal beam stress at the rim. The ring girders are proportioned for a combination of the bending stress and the stresses due to shearing and radial forces, all in accordance with Schorer's theory.[1] Stresses at the rim and in the ring girders should preferably not exceed one-half the yield point of the steel used.

The rockers are made of cast steel and are designed to maintain the penstock at the same level during temperature movements. This is accomplished by a radial surface in contact with the support columns of sufficient length to accommodate a temperature movement corresponding to a range of 150°F. plus some extra length for safety. The required length will also depend on the distance of the rocker support from the anchor, which varies from one support span to nearly one-half the distance between anchors, the latter distance being used in the design. Each rocker is provided with a steel tooth bolted to its side to keep it properly aligned by means of two pins placed in the bearing shoe. The width of the rocker must be sufficient to carry the load at a safe bearing pressure to the concrete pier. For the 72 in. diameter penstock of $1\frac{3}{16}$ in. shell thickness, the combined pipe and water load is about 84,000 lb. for each rocker. Determining the maximum safe capacity of the rocker with the empirical formula of $600\ d \times w$ (where d is the diameter of the rocker, or 36 in., and w is the width of the rocker, or 6 in.) the actual load is $\frac{84{,}000}{36 \times 6} =$ 388 lb. per linear inch of radial surface of the rocker, as compared to the permissible value of 600 lb.

Support Piers. Steel pipes installed above the ground are usually made self-supporting as the pipe is an efficient beam if its circular cross section is preserved by ring girders at the points of supports. The ring girders carry the pipe and water load, by means of rockers, to the concrete support piers which, in this case, are located at 60 ft. centres as shown in Fig. 1. Piers are designed to support the dead load and to resist the longitudinal frictional forces due to the temperature movements in the pipe. There are very little data available on the frictional resistance to movement at rocker supports. It is customary, therefore, to use the experimental results of tests made on rollers some years ago which were published by the Hydraulic Power Committee of the former National Electric Light Association. These early tests yielded frictional coefficients of around 5 per cent. for rollers in good condition. It is safe to assume that pinned rocker supports will provide more resistance than rollers. To obtain practical values which will prevail after the pipe line has been in service for years and the roller or rocker supports have become covered with dust, it is desirable to double the coefficients obtained during the test and use 0·10 for rollers and 0·15 for rockers.[2] Temperature forces acting on both sides of a support tend to compensate each other. As identical coefficients cannot be expected at each support, it is customary to assume a support in excellent condition on one side with a low coefficient of 5 per cent. and a badly deteriorated support on the other side with a frictional coefficient of 15 per cent. This results in a differential coefficient of 0·10 which is used in the design of the piers. Fig. 5 shows a typical pier designed for the 72 in. penstock on a 20° slope, with a spacing of 60 ft. along the slope of the pipe. The pipe and water load supported on the piers is 168,500 lb., which with a differential friction coefficient of 0·10 will develop a longitudinal force of 16,850 lb. acting in either the upstream and downstream direction. This force, when combined with the dead load due to the weight of the pipe and water acting normal to the pipe axis, plus the weight of the pier concrete and earthload, gives a resultant force which intersects the pier base in its middle third as shown in Fig. 5. This is a requirement in pier and also in some anchor designs to assure stability and provide resistance against overturn. With a total dead load of 202,500 lb. on the pier base, a unit bearing load of only 4,070 lb. per sq. ft. is obtained on the pier base, which for a soil of loose rock and gravel assumed in the analysis is considered to be safe.

Pier and anchor bases are generally placed a minimum of 4 ft. below the finished grade line of the soil to obtain better bearing and provide frost protection where necessary. All piers should be adequately reinforced for the forces acting on them, using about

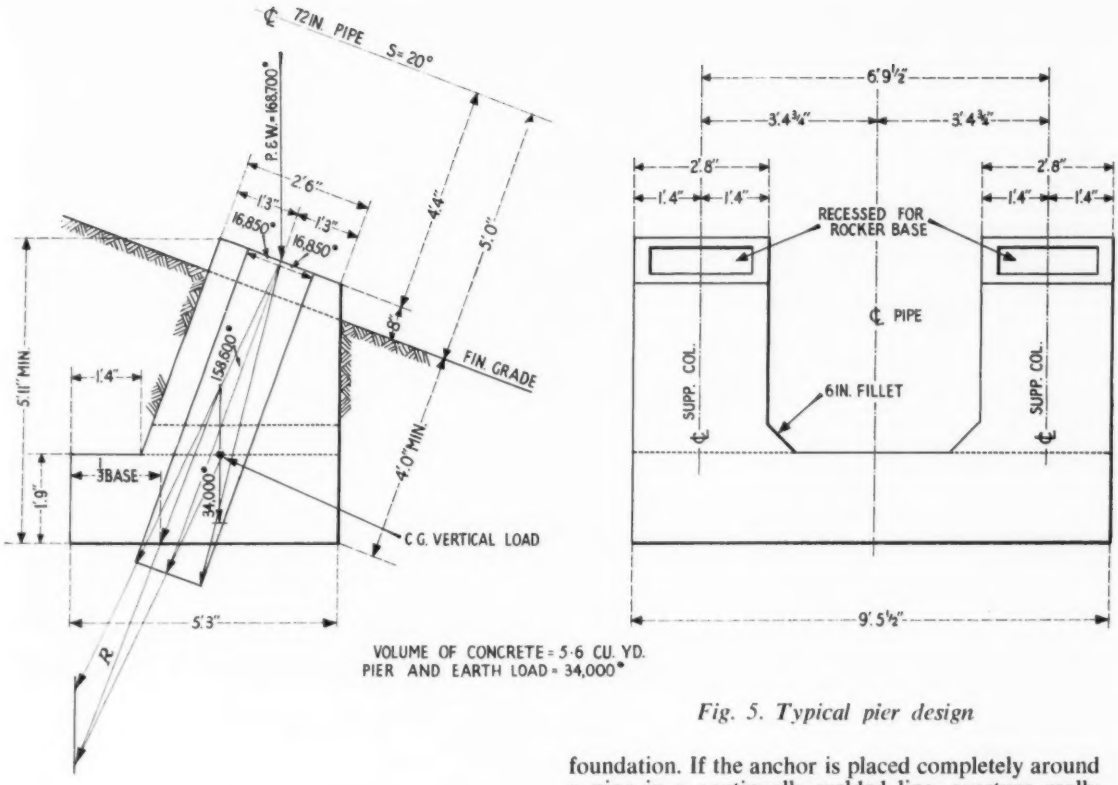
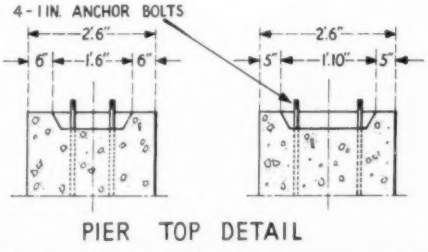


Fig. 5. Typical pier design



PIER TOP DETAIL

100 pounds of reinforcing steel per cu. yard of concrete. This reinforcement is not shown in the design. The tops of piers are recessed for the base castings of the rockers. Anchor bolts are provided for the base castings which will be grouted into the pier tops.

Concrete anchors. All welded steel pipe lines freely supported above the ground surface or in tunnels must be provided with anchors at bends and at intermediate points in long tangents. The purpose of the anchors is to fix the pipe line in place during installation and operation. They resist the various forces acting on the pipe or bends. During installation with the pipe empty, only the temperature and gravity forces need to be considered. The forces which are active on a pipe bend when the line is in operation consist of temperature forces, hydrostatic, dynamic and gravity forces. Anchors are generally not required for buried pipe except at horizontal bends with large deflection angles and at vertical or over-bends with high uplight forces which cannot be resisted by the backfill load. The combination of forces acting on an anchor[2] tend to overturn it or slide it along its soil foundation. If the anchor is placed completely around a pipe in a continually welded line, overturn really cannot take place. Such anchors should, therefore, be proportioned primarily to safely resist the sliding forces. Whenever possible anchors should be placed on a solid rock foundation, as this permits the use of a sliding coefficient of 0·65 as compared with a sliding coefficient of 0·50 for a soil consisting of loose rock and gravel and of 0·35 for common soil. This sliding coefficient will determine the required concrete volume of the anchor for a given force. Fig. 6 shows an anchor designed for a combined bend on the 72 in. penstock. The bend is completely enclosed in the anchor concrete. A resolution of forces for the expanding condition gives a combined force of 1,478,000 lb. in the upstream tangent acting in the downstream direction and 1,216,000 lb. in the downstream tangent acting upstream. A horizontal resultant of approximately 440,000 lb. and a vertical component of approximately 160,000 lb. are indicated. With a dead load (due to pipe and water) of 180,000 lb. supported on the anchor and an upward force of about 160,000 lb., the net downward force with the penstock under pressure is only about 20,000 lb. As the outer surfaces of the anchor are to be placed against undisturbed soil for which a sliding coefficient[2] of 1·00 can be used, the concrete volume will be determined as follows:

$$\frac{440{,}000 - 20{,}000}{1{\cdot}00} = 420{,}000/4{,}050 = 103{\cdot}5 \text{ cu. yards.}$$

Anchors are to be reinforced near the outside surfaces using approximately 75 lb. of steel bars per cu. yard of concrete, which is not shown in the design.

Fabrication and Installation. Steel pipe in sizes which can be shipped by rail or truck (limited to about 12 ft. diameter in the United States) is usually

fabricated in shipping or erection lengths varying from 20 to 60 ft. In this analysis shipping lengths of 40 ft. are used. The pipe sections are completely fabricated in the shop within specified tolerances and with edges of plates 1 in. and less in thickness prepared for field welding. Pipe sections with plate thicknesses exceeding 1 in. which, according to the code, cannot be welded without stress relieving, have riveted field girth joints. The holes for the portion of the rivets which can be driven in the shop are drilled and reamed to size and those for the field rivets are sub-drilled in the shop and reamed to size in the field.

All shop welding where possible is performed on automatic machines and all shop and field welds in the pipe shell are radiographed. Before radiographing any weld ripples and surface irregularities are removed on both sides to improve radiographic contrast. Unacceptable defects are removed and replaced with sound welds. Repaired welds are re-radiographed to prove the quality of the repairs. Pipe sections fabricated from plates over 1 in. in thickness are stress relieved using a furnace temperature of from 1,100° to 1,200°F., applied at the rate prescribed in the code. After radiographic inspection, each completed pipe section is subjected to a hydrostatic pressure test at 150% of the design pressure. This test is made in accordance with the code except that the hammer test is omitted. Any defects in the plate or welds which are disclosed during this pressure test are repaired and again radiographed. If these repairs are of a major nature, as determined by the inspector, the affected pipe section is required to be stress relieved again and subjected to a further pressure test. Before shipment all pipe sections are marked and match-marked for field assembly in accordance with an erection diagram supplied by the contractor. Each pipe section is painted on the inside in the shop, after ensuring that the surfaces are blasted clean to bright metal. First a coal-tar primer is sprayed on, which is followed by a coat of hot coal-tar enamel, applied by spinning in accordance with AWWA standard specifications. At the field girth joints, the coating is omitted for about 12 in. to facilitate welding or riveting. After the excavation has been completed and the intake structure and support piers constructed, the penstock is installed in accordance with the drawings and erection diagrams furnished by the contractor. Each pipe section is placed accurately to line and grade. The bend sections will be installed first and fixed in place with the concrete anchors before proceeding with the penstock tangents between the anchors. Installation will proceed from the anchors toward the expansion joints. While the support piers must be constructed before the pipe sections are installed, the concrete anchors, or at least the top portion around the pipe, will be constructed after the pipe is in place. The bases of piers, anchors and other concrete structures are placed on undisturbed soil and the backfill around the completed structures is placed and compacted in accordance with applicable AWWA specifications. Welding of the field girth joints is performed manually, using double-welded butt joints in the pipe shell. The rivets in the riveted field girth joints will be driven after the pipe sections are properly aligned. Before riveting, the rivet holes are reamed to size and chamfered on the outside of the butt strap. All caulking is done after riveting.

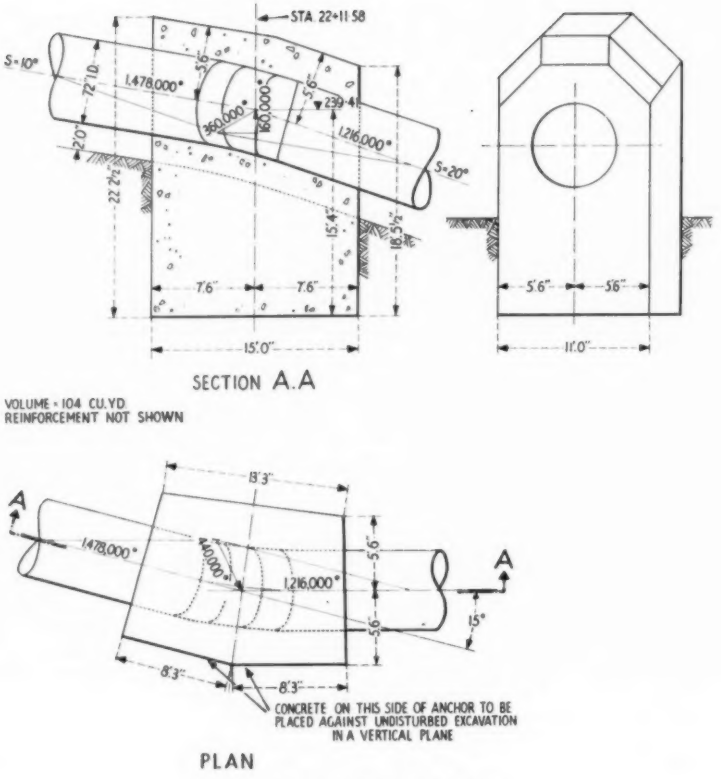


Fig. 6. Typical anchor design

After installation and before painting, the penstock is filled with water and subjected to a proof hydrostatic test primarily to check the watertightness of the expansion joints and the riveted field joints. A pressure equal to about 150% of the design pressure is applied at the downstream end and maintained long enough to permit the inspection of all joints. Any leakage in expansion joints is eliminated by further tightening of the gland bolts or by the addition of more packing. In the riveted joints any leakage is eliminated by caulking.

After the proof hydrostatic test, the interior coating omitted at the welded joints is applied and the shop coating on the inside of the penstock is repaired where it has been damaged during shipment or installation. The outside surfaces will then be blasted to bright metal and painted with one coat of synthetic red lead followed by two coats of aluminium paint.

(To be continued)

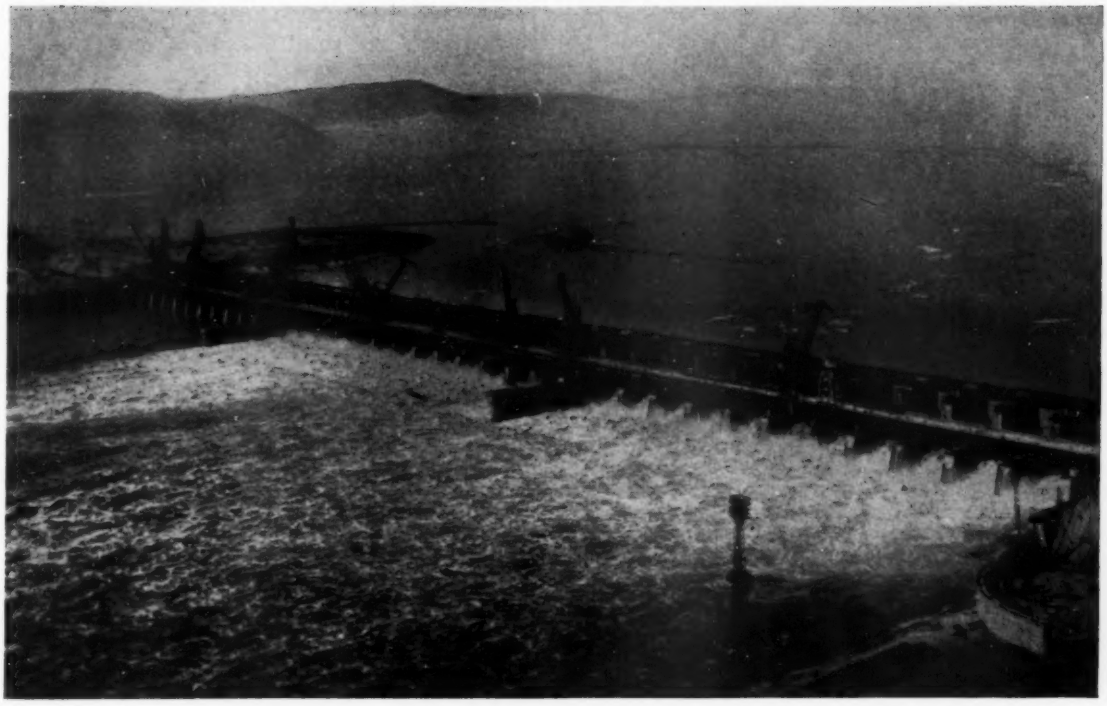


Fig. 9. Work in progress on spillway block during spring flood of 1956

Volga River Development

Brief particulars are given of the scope of this development scheme with special reference to the Kuibyshev Power Station

By ENGINEER G. A. RUSSO

PART TWO

AT the first stage, in 1951, cofferdams were built around the foundation pits of the power house on the right bank, the spillway block and navigation locks on the left bank, and all the necessary temporary buildings were erected. At this period, the Volga flowed along its main bed, which continued to be used for navigation.

During the second stage, between 1952 and 1955, work proceeded on the power house, the concrete-spillway section, the navigation locks and parts of the earthfill dam in the inundated area of the river. In October, 1955, when the power house block was ready to pass the major part of the Volga discharge, and navigation could be diverted to the locks and the canal connecting them, the right-bank cofferdam was dismantled and the Volga bed was closed by a dyke of random rubble, whereupon work on the earthfill dam proper started. During November and December, 1955, the level of the water upstream of the installations rose 10 m.; the first generating sets of the power house were put into operation at the end of December.

In the winter of 1955/1956, work on the spillway block was intensified in order to have the structure ready in time for the 1956 spring floods. The flood flow was passed by the power house and spillway blocks and the level of the upstream water rose by 9 m.

By the spring of 1957, all the permanent main works had been completed and the stored water had reached the planned level. The construction of the whole plant and ancillary works involved the following operations:—

Excavation and earthfill work ...	197 million cu. m.
Concreting work	7·55 million cu. m.
Total weight of steel piles driven ...	44,000 tons

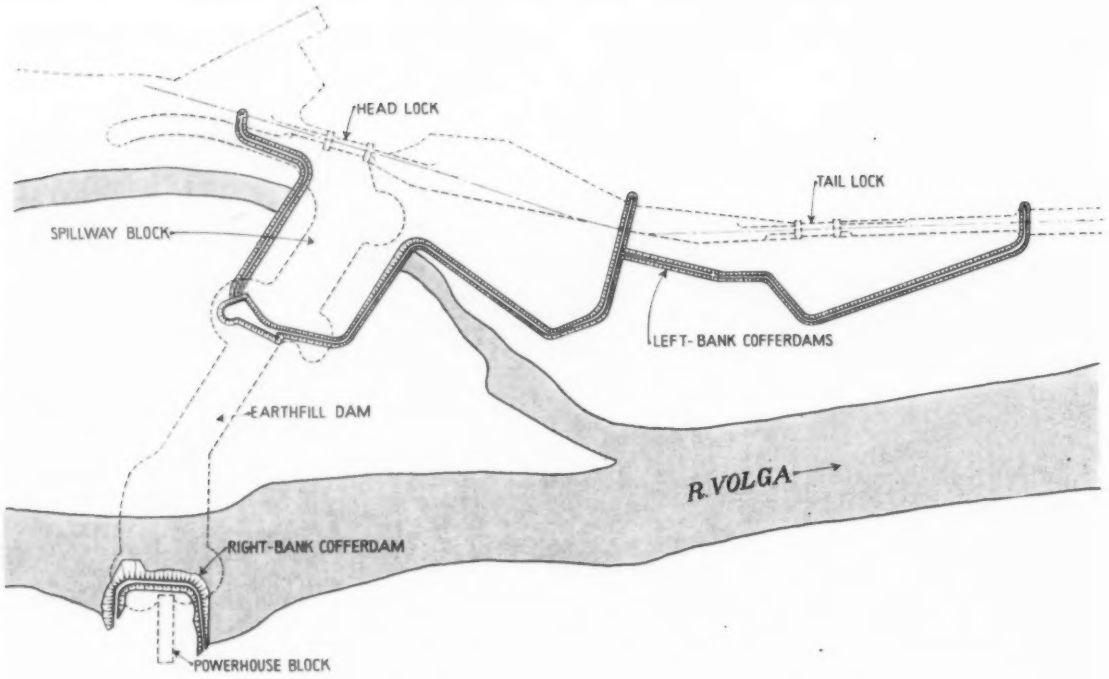


Fig. 10. Plan of Kuibyshev site showing layout of cofferdams

Stone reinforcement of earth-dam slopes	3·84 million cu. m.
Drainage blankets and filtering material	1·51 million cu. m.
Aggregate weight of structural steel and mechanical equipment ...	236,000 tons

Work on the Kuibyshev installations began towards the end of 1950, earthwork was in full swing by 1952, and concreting work by 1953.

Construction work reached its peak in 1955, when no less than 46·5 million of cu. m. of earth work was effected and 3·12 million of concrete and reinforced concrete was placed. The most extensive operation

Fig. 11. Tail-lock entrance to canal

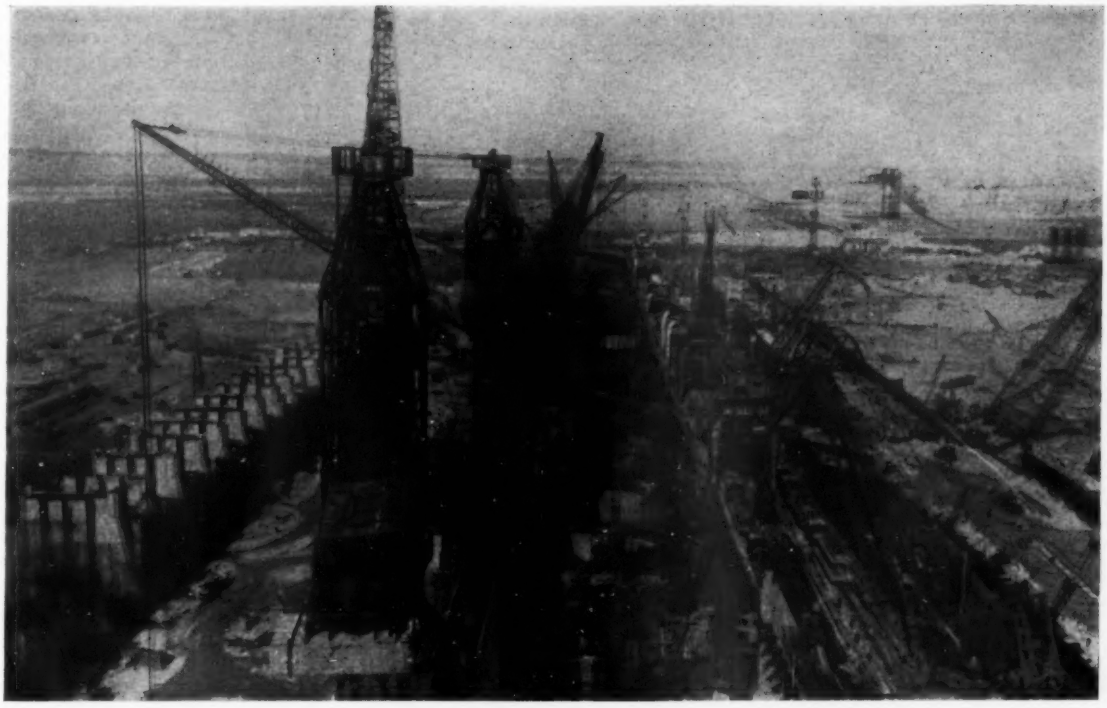


Fig. 12. Concreting in progress on power-house block

of all was the excavation of the foundation pits for the power-house and spillway blocks, together with the erection of the earthfill dam.

The excavation pit for the power house block covered an area of about 40 hectares (about 100 acres), and entailed the extraction of 8 million cu. m. of sandy clay, lenses of clay mixed with sand, gravel and stones, and 1 million cu. m. of sandy soil. The foundation pit reached a depth of 45 m. at the river bank and 20 m. in the river bed. Mixed soils of sand and gravel were extracted by means of power shovels and suction dredgers before work on the earth dam proper started. Clayey soils were handled by electric excavators fitted with 3 cu. m. buckets. A fleet of 10 ton self-dumping trucks attended to the transport of the earth needed for the levelling of the building site, embankment work and dumping. The volume of material dug out monthly reached 700,000 cu. m. in 1953. The excavated pits were dewatered by pumping; pump borings were put down to a depth of 70 m. in places and 280 deep-well pumping units were used in the operation.

The excavation covered an area of 70 hectares (about 173 acres) and was 30 m. deep. The necessary excavation work was carried out by two types of dredger, one with a capacity of 1,000 cu. m. of dredged material per hour and a suction lift of 80 m., and the other having a capacity of 500 cu. m. per hour and a lift of 60 m. Corrections to the bottom of the excavation were effected by means of 3 cu. m. power shovels. The level of the water table under the foundation pit of the spillway block was lowered by pumping, for which purpose 300 holes scattered over the whole area were bored.

The fill material of the earth dam, totalling 24 million cu. m. in volume, was placed by means of section dredgers of 1,000 cu. m. per hour capacity and capable of operating with a lift of 60 m. In the river bed itself the bottom part of the dam was built up under water. Once the lower part of the dam had emerged from the water, filling progressed at the rate of 80 to 100 cm. a day. About 65% of the earthwork carried out for the dam was effected by suction dredgers, 6% by scooping and 29% by excavators, scrapers and bulldozers. A large proportion of the excavated soil was used for embankment work.

The aggregates required for the concrete were ob-

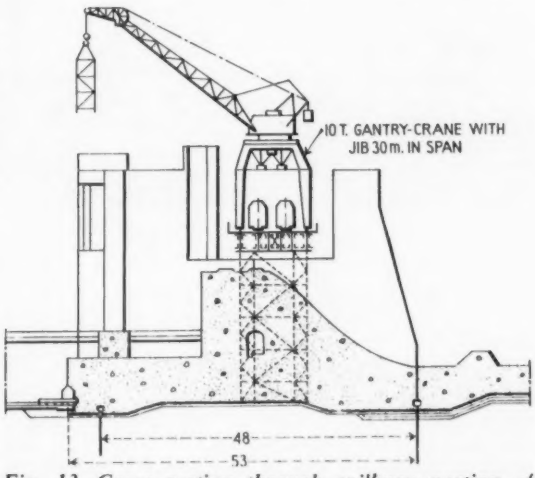


Fig. 13. Cross section through spillway portion of dam

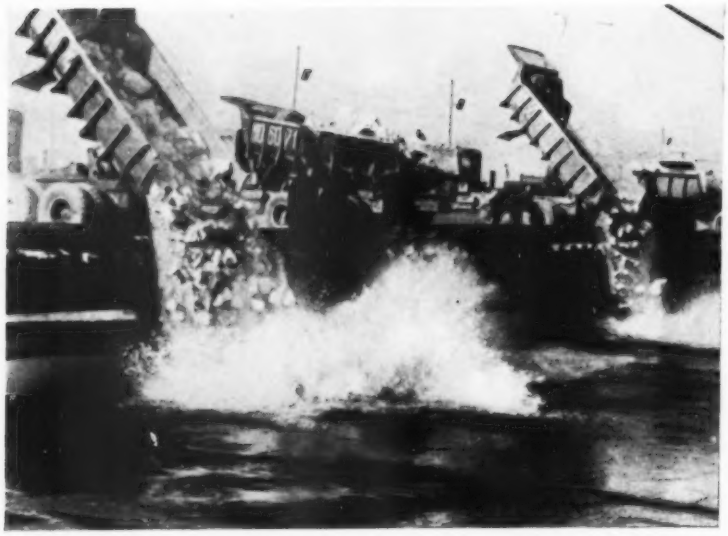


Fig. 14. Dumping trucks being discharged into earthfill section of dam

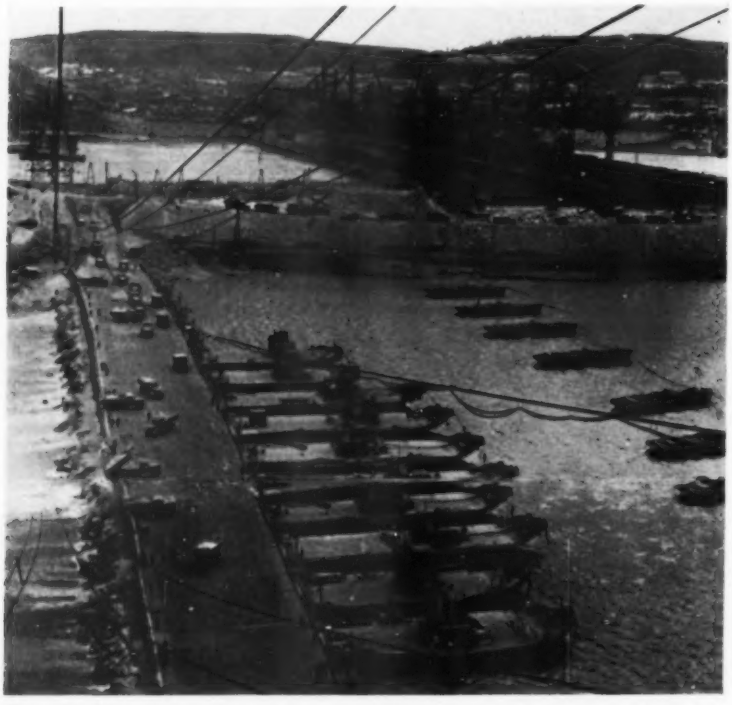


Fig. 15. Work in progress on earthfill dam; power-house block in background

tained from local deposits. Gravels and stones were crushed in five quarries at the rate of 5·2 million cu. m. of material a year. The physical and mechanical properties of the sandstone and dolomite quarried for construction work differed very widely, compressive strengths varying between 400 and 160 kg. per sq. cm., specific weight between 2·3 and 2·7, and porosity between 4 and 18%. The fine and medium sand used for the concrete was also obtained locally and varied in granulation between 1·5 and 1·8 mm. The small proportion of coarse sand (1·8 to 2·2 mm.) required was drawn from the southern end of the Samara loop of the river. Winter temperatures in the region of the Kuibyshev plant sink as low as −49°C., temperatures below zero occurring about 80 times in the course of a year. To meet these severe temperature conditions and to deal with the fluctuations in water level caused by the daily regulation downstream of the works, all submerged structures, as well as all parts subjected alternatively to the action of water and frost, were faced with concrete slabs, 8 cm. thick, made from gravel or fine aggregates obtained from crushed rock.

The batching plant set up on the right bank of the river was equipped with 16 mixers, having a total capacity of 120 cu. m. per hour, a plant for the preparation of reinforcing steel with an output of 200 tons per shift, a plant for manufacturing concrete slabs and ferro-concrete parts, stores, and equipment for heating the concrete components during the cold weather. On the left bank of the river there were two batching plants, one supplying the spillway block and the other the navigation locks. These plants were similarly equipped to those on the right bank. The total monthly output of all batching plants was about 400,000 cu. m.

The steel bridge, 40 m. high, erected for the construction of the power house, Fig. 12, carried three railway tracks and a track for a 10 ton portal crane fitted with a jib having a 30 m. range. Two parallel 15 ton cableways with a span of 390 m. were also in service. The concreting of the spillway block was also carried out with the help of a bridge on which two railway tracks ran between the rails of the gantry cranes (Fig. 9).

The reinforcing steel, shuttering and the facing slabs were handled by small tower cranes which were moved from one working point to the other by means of the gantry cranes.

Concreting work for the navigation locks was mostly done by concrete pumps having an output of 40 cu. m. per hour. Transport from the batching plants to the pumping units was effected partly by self-dumping trucks and partly by belt conveyors. Reinforcing steel, shuttering and facing slabs were handled by cable cranes and 2·5 ton mobile cranes. The largest volume of concrete placed in the course

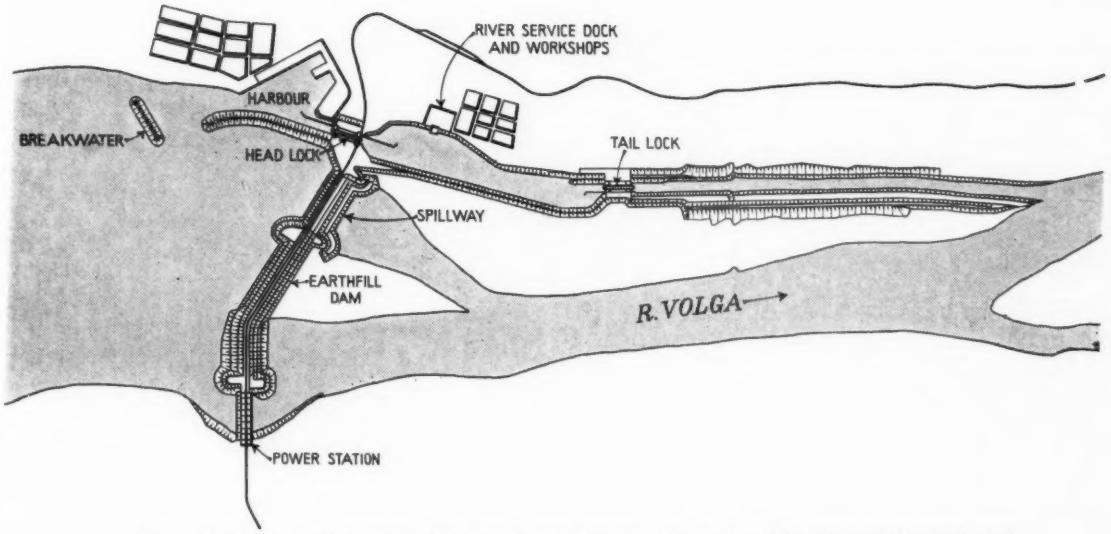


Fig. 16. Layout of construction work for power house, earthfill dam and canal lock

of a single month was 357,000 tons, the record output for a single day being 19,050 cu. m. in August, 1955.

The most awkward stage in the construction of the Kuibyshev plant was the closing of the Volga river channel. The first move was to erect in the middle of the river a cableway tower which divided it into two sections. In the summer and autumn of 1954, barges kept dumping stones into the river; the bottom part of the earth dam was built up on a width of 100 m. of the river bed, to reduce undermining. Flood discharge reached 40,000 cu. m. per sec. in 1955, the flow velocity being 3·0 to 3·5 m. per sec. along the shore lines and 3·5 to 4·0 m. per sec. around the cableway tower.

In spite of these conditions navigation proceeded normally and any scouring of the foundations of the earthfill dam was negligible. After the 1955 floods, a fleet of barges and the cableway were both engaged in building up the bank of the earth dam in the left-hand section of the river bed, which was utimately closed on August 14. The downstream locks were open to navigation at the beginning of August, Fig. 11, and the whole flow of the Volga was diverted through the 330 m. right-bank section of the river.

Work on the dam bank in the right-hand section of the river bed was begun by dumping rock from the cableway before the cofferdam of the power house was dismantled in August and September. To complete the closure of the whole river bed, a pontoon-bridge was assembled (Fig. 15), and subsequently opened to traffic on October 22, at which date stone dumping began along the whole length (Fig. 14). On that day the level of the water was within 20 cm. of the dam top, and the river flow was 3,600 cu. m. per second. On October 24, the flooding of the power-house pit began, and exactly a week later the closure of the river bed had been completed. The maximum height of the bank above water had reached 193 cm. The material dumped into the river comprised 1,765 concrete pyramids and 750 concrete cubes weighing up to 3 tons each, and 64,000 cu. m. of stones. Flow velocity downstream of the embankment was 5·0 to 5·5 m. per second; no scouring of any importance was observed in the downstream section of the river.

It should be recorded that hydrological conditions in October, 1955, were most favourable to the closing of the river bed; in fact, the discharge remained below 4,000 cu. m. per sec. at that time, whereas in other years the volume of the discharge may be as much as 12,000 cu. m. per second. Another welcome factor was the retarded appearance of floating ice in the autumn. It may be said that, on the whole, the building of the earth dam did not meet with unusual difficulties.

The Kuibyshev hydro-electric plant is now in full operation. Two floods, including the great spring flood of 1957, have been passed by the spillway dam and the power-house block with a 10% safety margin. Navigation through the locks and their connecting canal is now in full swing, and the hydraulic drives for the lock gates are giving every satisfaction. By December 1, 1957, the 20 generating units of the power station had supplied over 9,000 million kWh to central Russia and the Volga basin. The 400 kV transmission line is now in operation and recent research work has established the possibility of a further increase in the voltage to increase its capacity.

Sixth International Congress on Large Dams. A Bulletin has been issued by the International Commission of the World Power Conference to explain the final arrangements which have been made for the Sixth Congress on Large Dams, which will be held in New York City from September 15 to 20, 1958. Issued with this Bulletin is a Congress Registration Form and a booklet prepared by Thos. Cook & Son Limited through whom members are recommended to make their travel arrangements. This booklet contains three reservation forms applicable to the study tours, for hotel rooms and for special events in New York.

A list of the 12 papers to be presented by the British National Committee can also be obtained from the Secretary, British National Committee on Large Dams, 201-2, Grand Buildings, Trafalgar Square, London, W.C.2.

The Kariba Flood

Fig. 1. A view of the site two months before the flood

THIS year's flood at Kariba reached a peak of 570,000 cusecs—almost twice as much as last year, which at 290,000 cusecs was one of the highest flows previously recorded.

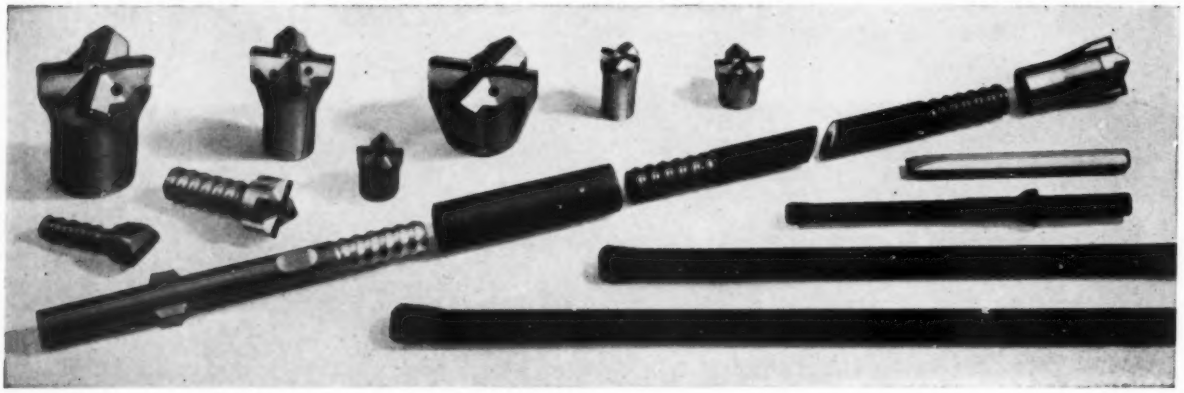
The annual flood at Kariba has two distinct components: that derived from the upper reaches in Angola and Northern Rhodesia, and the run-off from the catchment between Livingstone and Kariba. The peak of the former, which contributes the great bulk of the three-months' flood period, normally reaches the dam site between the middle of March and the end of April after the flashy floods due to the local rains have subsided. This year the two components coincided, due to the very early arrival of the main flood, and prolonged and heavy rains on the catchments just above Kariba. At the peak, it is estimated that 40% of the river flow came from the catchment between Kariba and Livingstone.

About three weeks advance warning of the arrival of the main flood was given at the beginning of February by the Hydrological Branch of the Irrigation Division of the Southern Rhodesian Government, when it was reported that the upper reaches had started to rise some two months ahead of their characteristic annual pattern. As the lower catchment flood depends on local rains, no appreciable advance warning of this component was possible.

By the 20th February the flow had exceeded last year's peak and was still rising fast. On the 21st, the construction road bridge was swept away, and by the 26th a peak was reached at an estimated discharge of 460,000 cusecs, after which the river began to fall. It seemed as if the worst was over, but on 1st March came the news that all the upstream gauges were again rising fast. The flood swept down on Kariba once more, carrying away the suspension footbridge which had already been raised 12 ft., and overtopping the main cofferdam. A peak of 570,000 cusecs was finally reached on 5th March, when the upstream water level was 10 ft. higher than the top of the cofferdam (which had also been raised 12 ft.) and only 4 ft. below the sill of the low-level power-station intakes, upon which construction is in progress. Downstream of the cofferdam, construction roads had been badly eroded, and temporary buildings housing such plant as the chillers for cooling the concrete in the dam, and contractor's offices were threatened or carried away. Bulkheads were built protecting the tunnels giving access to the underground works.

After the floods had receded it was possible to assess the damage. Most of the construction plant had been taken to safety. The footbridge was restored five weeks after its loss, and a new roadbridge should be in service in two or three months' time; mean-

Fig. 2. The peak flood of 570,000 cusecs at Kariba dam site on 5th March, 1958. This view was taken from practically the same point as the earlier photograph reproduced in Fig. 1 on the opposite page



SANDVIK COROMANT—

A Complete Range of Drill Steel Equipment

You know as well as we do the advantages of buying all your drilling equipment from one supplier. These advantages become still more evident if you buy from the Sandvik Range. The Sandvik Steel Works are the world's largest manufacturers of tungsten carbide for rock drilling. Their production covers integral steels, detachable bits, extension steels and stone working tools—all made of high-quality Swedish alloy steel, all fitted with the well-known Coromant tungsten carbide inserts.

Integral steels with 50% longer life

Sandvik Coromant integral steels have up to 50% longer rod life than ordinary steels, thanks to anti-corrosion SR-treatment, which protects them during transport, storage and actual drilling. In addition, air-tight plastic caps give bit and shank extra protection during transport and storage. They are available in these standard sizes:—

$\frac{3}{4}''$ hollow hexagon	1′4″-13′1″
$\frac{7}{8}''$ hollow hexagon	1′4″-21′0″
1″ hollow hexagon	2′6″-21′0″
Flexible drill steels	2′7″-31′6″

Precision-made rock bits

The threads of Sandvik Coromant (cross and X-design) bits are precision milled. The bits are so accurately manufactured that not only smoother drilling but *longer life* are ensured. Standard bit diameter sizes range from $1\frac{1}{2}''$ to $4\frac{1}{2}''$. The 773 bits (bottoming type) are available with GD400 and GD600 thread, or with $1\frac{1}{4}''$, $1\frac{1}{2}''$ and 2″ rope thread. The 776 bits, for standard shoulder-type drill rods, are available with threads ranging from $\frac{7}{8}''$ to $1\frac{11}{16}''$.

Efficient extension steels

The rope-threaded joints of Sandvik Coromant extension steels are solid and make joining and unscrewing extremely easy. Sizes available: $\frac{7}{8}''$ and 1″ hexagon steels, $1\frac{1}{4}''$ and 2″ round steels. A special feature of the $1\frac{1}{4}''$ equipment is the $\frac{1}{2}''$ flushing hole, about twice as large as most. This gives better cleaning of the bore hole and a higher rate of advance, reduces wear and risk of steels sticking. The 'cold rolling' technique makes this wider flushing hole possible *without any loss of strength.*

Wide variety of Stone Working Tools

A single plug hole steel made by Sandvik is capable of drilling up to 1000 holes, each about 3.9″. Sandvik Chisel Steels are made with rubber sleeves to reduce vibration and protect the worker. Sizes available: Plug Hole Drill Steels with bit diameters ranging from approx. $\frac{21}{32}''$ to $\frac{7}{8}''$. Chisel steels with bit diameters from approx. $\frac{5}{16}''$ to $\frac{25}{32}''$.

The World's foremost drilling unit

Sandvik Coromant extension and drill steels have been developed in close co-operation with Atlas Copco, manufacturers of rock drills and other compressed air equipment. The combination of Sandvik steels and Atlas Copco rock drills is the world's most widely used drilling unit—responsible for the drilling of more than one thousand million feet each year!

The Sandvik Coromant accessories featured here are sold and serviced throughout the world by the Atlas Copco Group of Companies. We have mentioned only the most important products and sizes: for further details, please contact your local Atlas Copco Company or Agent, or write to Atlas Copco AB, Stockholm 1, Sweden.

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DS577

while the blondin cableways maintain communications across the river. The cofferdam was protected from damage when overtopped by prepared sections in the downstream wall which were designed to fall outwards, thus relieving the reverse pressure on the arch. It had always been anticipated that overtopping might occur and it is not expected that any damage has occurred to the permanent structures.

Work has already started to a revised programme which should enable the completion of the main dam and commissioning of turbo-generators to maintain the original programme. It is heartening that in spite of the setbacks of this extraordinary flood the first parts of turbine No. 1 were brought into the power station on 17th March, that is a fortnight ahead of schedule.

Rapid Concrete Mixing

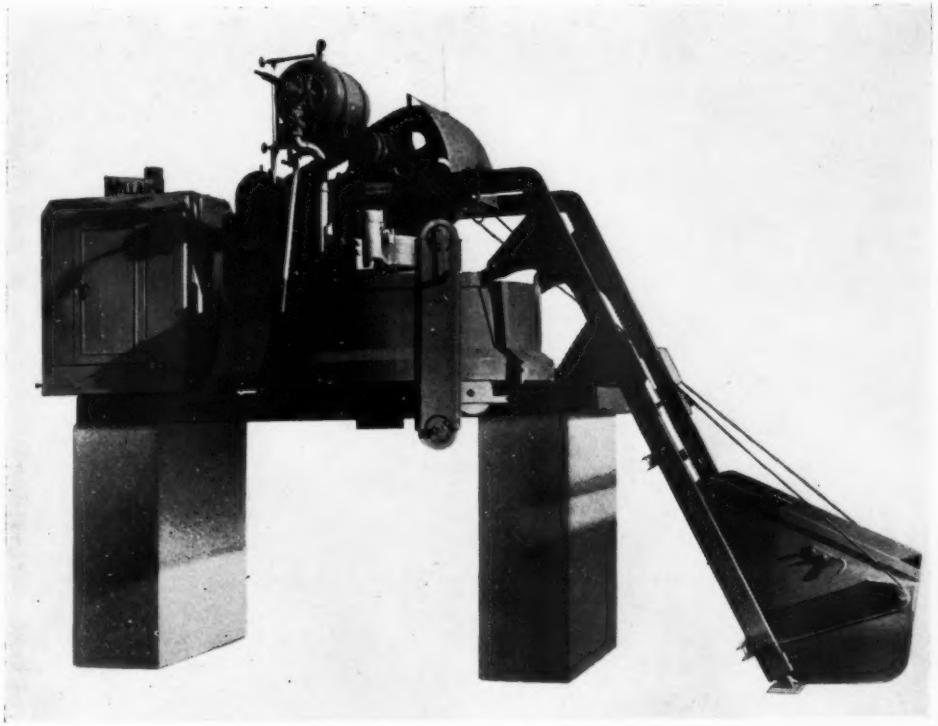

Fig. 1. Millars' 30-second mix Universal Concrete Mixer

A novel design of concrete mixer is now in production at the Thorley Works, Bishop's Stortford, of Millars' Machinery Co. Ltd. This machine is said to be able to mix a 10 cu. ft. batch of concrete, of any composition or consistency, in 30 sec., and a recent demonstration we attended convinced us that this claim was justified. The value of this short mixing time in terms of output needs no emphasis. As an actual instance we were told of a new construction at a British colliery where an eight-hour concreting shift had previously been worked with a normal ½ cu. yard mixer. By replacing this machine with a Millars' 10 cu. ft. mixer of the new design the same amount of concrete has since been mixed in five hours.

A view of the machine appears in Fig. 1, and the interior of the mixing pan is shown in Fig. 2. The pan is of the open cylindrical type, and is fabricated from steel plate with renewable lining plates. It rotates horizontally round its centre point and carries the material in a clockwise direction. Three flexibly

Fig. 2. A view inside the mixing pan showing the eccentrically mounted mixing blades, the side and floor scrapers, and the discharge door which is shown partly open

For hard 3-shift running

but easy installation!

A set of three AR3-L compressors, skid-mounted with diesel drive, being installed at one of the mines of the Union Minière du Haut Katanga, in the Belgian Congo.

SPECIALLY DESIGNED TO OPERATE where cooling water is scarce are the Atlas Copco AR-L compressors. They are absolutely self-contained, with a closed-circuit cooling system. Normally they are skidmounted and complete with electric motors, but—as in the instance shown above—they can be operated by diesel engine. These AR-L compressors are easy to install; no concrete foundations are needed, nor any permanent water supply.

Portability plus highest efficiency

Atlas Copco AR-L compressors combine the advantages of water-cooled machines—high efficiency and low discharge air temperature—with those of air-cooled models—easy installation and maintenance. They are cooled by a fan-cooled intercooler and a radiator for the cylinder water. The complete cooling system is a part of the compressor unit, entirely self-contained.

DATA ON AR-L COMPRESSORS

Type	Maximum pressure in psi	Speed in rpm	Capacity in cfm at 100 psi	Power required at 100 psi in hp	Total weight in lbs.
AR1L	125	730	381	80	2,900
AR3L	125	585	646	130	5,300

World-wide sales and service

The Atlas Copco Group puts compressed air to work for the world. It is the largest group of companies specialising solely in the development and manufacture of compressed air equipment. It embraces Atlas Copco companies or agents manufacturing or selling and servicing Atlas Copco equipment in ninety countries throughout the world. For further details contact your local Atlas Copco company or agent, or write to Atlas Copco AB, Stockholm 1, Sweden, or Atlas Copco (Great Britain) Limited, Beresford Avenue, Wembley, Middlesex.

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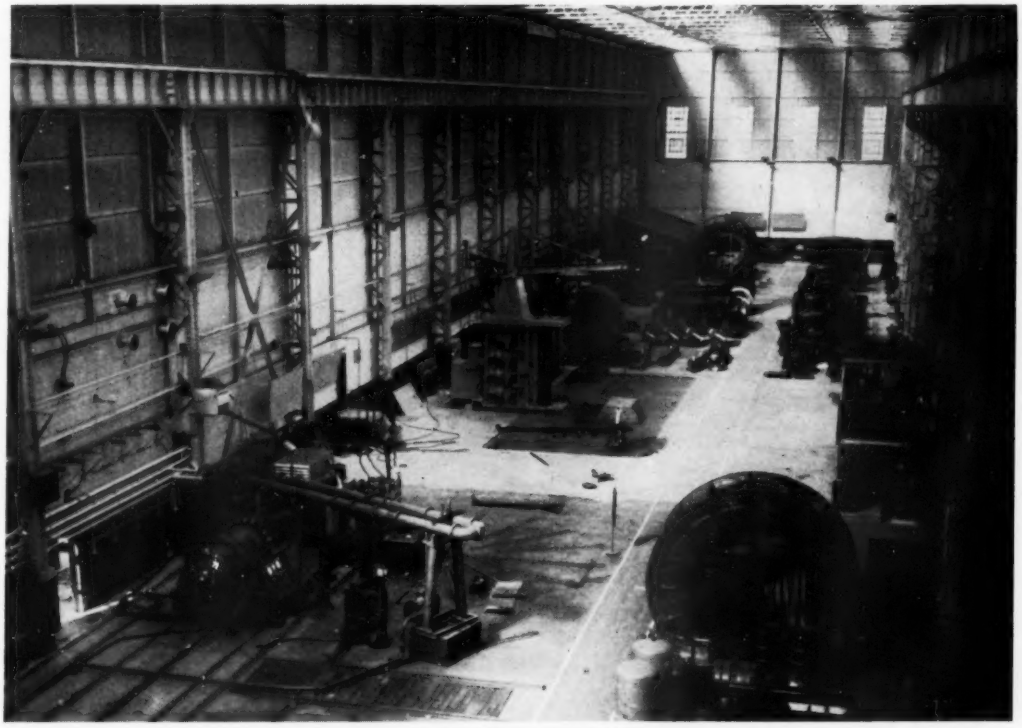
mounted steel mixing blades are suspended eccentrically within the pan and rotate in the opposite direction. By means of this counterflow action the blades describe spiral paths in the batch and during each cycle cover every point in the base. Flexibly mounted wall and base scrapers remove any accumulation from the wall and base of the pan and impel the material inwards to the mixing blades.

In the centre of the floor of the pan is a circular discharge door which is normally flush with the floor and rotates with it. To discharge the batch a hand control lowers the door and swings it to one side, upon which the revolving scrapers impel the batch through the hole.

A charging skip is incorporated in the machine, and a regulated quantity of water is added to each batch from a 14 gal. semi-automatic syphon-type tank.

Drive can be by diesel engine or by electric motor.

Harland's New Water-Turbine Shop

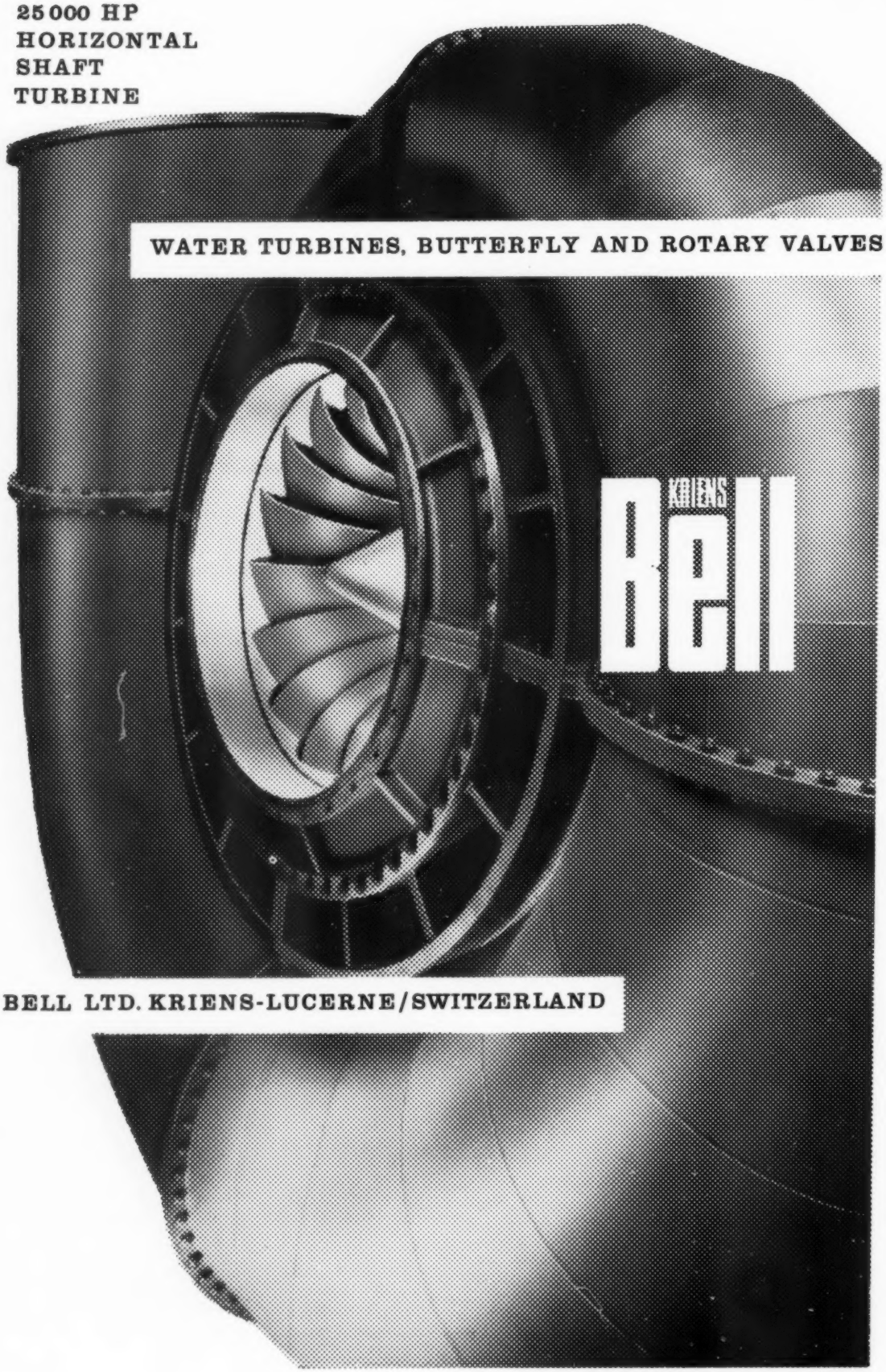


The new water-turbine shop of The Harland Engineering Co. Ltd., Alloa

The Harland Engineering Co. Ltd., established in a modest sort of way in 1903, had already earned an enviable reputation for medium-heavy and specialist hydraulic plant by the end of the 1939–45 war when they decided to take up the manufacture of water turbines—a move which received great stimulus by the decision to develop the considerable hydro-electric potential of the Scottish Highlands under the aegis of the North of Scotland Hydro-Electric Board. Up to date the firm have now manufactured over 220,000 h.p. of water turbines of various kinds, some 25% of which have been exported. This equipment has been manufactured under a licence agreement with the S. Morgan Smith Company of Pennsylvania, whose extensive experience in this field is exemplified by the fact that they have produced turbines to a total power of 17 million h.p. in units of up to 140,000 h.p. The increasing demand for hydro plant has caused the Harland Engineering Company to build a new water-turbine shop and this we were invited to inspect on April 29, when it was officially opened by The Rt. Hon. Thomas Johnston, C.H., LL.D., the Chairman of the North of Scotland Hydro-Electric Board. In the course of his speech, Mr. Johnston stated that the Board had so far spent £3¾ million on water turbines up to date, and that the experience gained by British manufacturers in supplying this plant had led to successful bidding for important export orders. With the increasing need for pumped-storage plants in the British network, he added, the field was opened for the development of suitable pumps for taking their part in this important work. The accompanying illustration gives a photographic view of the new shop.

Ferranti Pegasus Computer. Ferranti Limited have issued a second edition of List DC.10A which describes the Pegasus computer which is fitted with magnetic tape equipment.

25 000 HP
HORIZONTAL
SHAFT
TURBINE



Abstracts from the World Technical Press

Surge-Tank Studies

This paper describes the tests effected on the surge tank of the Cordeac plant (Drac Development) with special relation to Thoma's condition. Following a short description of the plant, the tests are reviewed and the authors deal at length with Thoma's condition. These tests have enabled a fair comparison to be made between the actual operating stability of a power house equipped with a surge tank the cross-sectional area of which is about equal to the Thoma area, and the stability range as deduced from the hypotheses on which the calculation of Thoma's condition is based. The tests have also shown the substantial influence of any delays intervening in the governing cycle of the power house. By taking these delays into account, it has been possible to verify the validity of the calculation hypotheses, and also to give a possible explanation of the character of the tests results obtained by Professor Scimemi on industrial installations. (J. Chevalier and M. Hug, *La Houille Blanche*, Vol. 12, December 1957, p. 888, 15 pp., 3 ff., 9 graphs.)

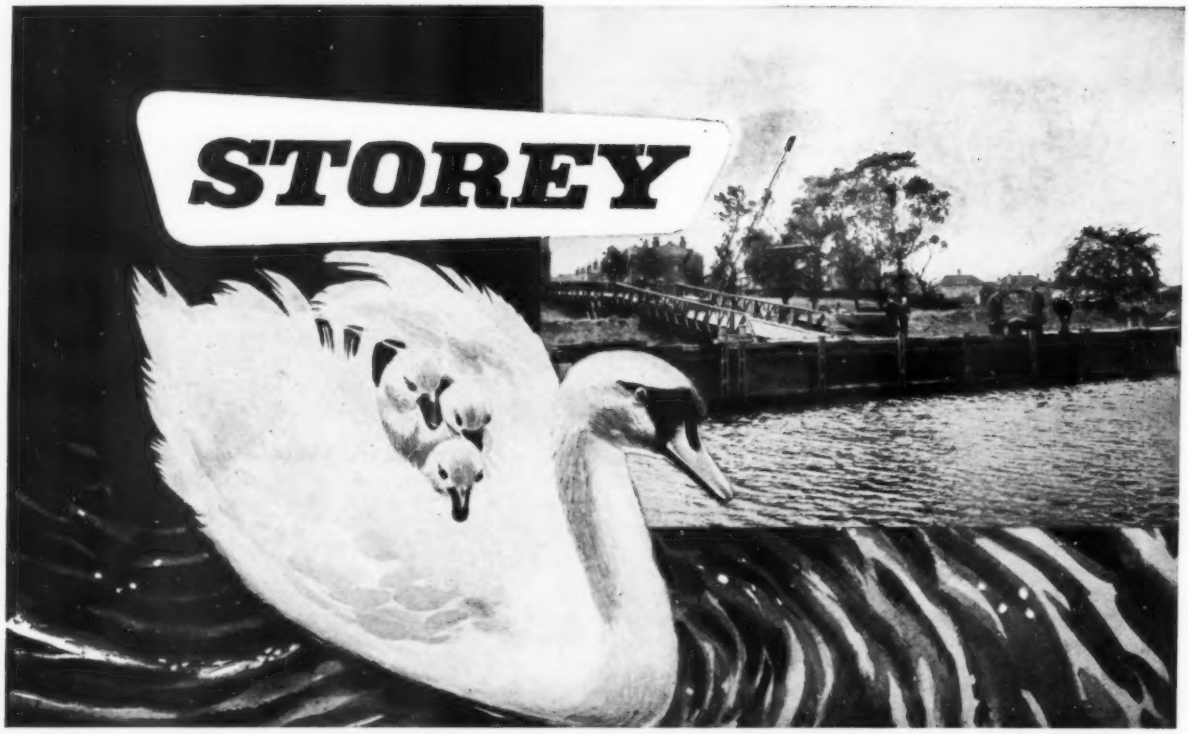
Thermal Behaviour of Concrete Dams

The author presents a theoretical analysis of temperature effects during the construction of concrete dams, and cites actual figures obtained from important Portuguese dams. Three groups of factors exercise the major influence on the thermal behaviour of a concrete dam: climatic conditions, the properties of the materials employed, and the methods of construction. Climatic conditions include the temperature of the air, the amount of wind movement, the temperature of the water, and solar radiation. The complex effect of water temperature, which is usually associated with variations in water level, is demonstrated by a series of curves from readings taken in Cabril reservoir, and the marked effect of solar radiation is shown by curves of temperatures taken 1 m. from Cabril dam crest during days representing the four seasons of the year. In discussing properties of materials importance is attached to thermal diffusibility, this being a term which represents the conductivity divided by the product of the specific heat and the specific weight.

The effects of temperature, type of aggregate, proportion of water, and type of cement on the thermal properties of the concrete are set out, and detailed calculations are given for the hydration temperature of concrete according to the methods of Carlson and of Schmidt. Under methods of construction the effects of the size of block, thickness of lift, and frequency of pouring are discussed, and various methods of artificial cooling are considered, including the cooling of the cement, the sand and the aggregate, and the admixture of ice with the water. The use of serpentines of pipes embedded in the concrete in which refrigerated water is circulated is considered to be one of the most effective methods of artificial cooling. The second half of the article is devoted to a full discussion of the application of this method to the temperature control of the recently completed dam at Picote, actual results being given in copious tables and charts. (António Ferreira da Silveira, *Electricidade*, No. 4, October–December 1957, p. 25, 28 pp., 26 ff., 8 tt.)

Grande Dixence Development

A full description is given of this great Swiss scheme, the purpose of which is to produce valuable peak-period energy by diverting into the existing Valdes-Dix reservoir the waters of the upper Herens and Zermatt valleys for which no local storage accommodation is available. The water supply network of the scheme, and the geological conditions at the reservoir and dam sites are discussed in detail, and the selection of a gravity instead of an arched structure is expounded. An interesting feature of the scheme is that the old 85 m. high dam of the S.A. l'Energie de l'Ouest Suisse (E.O.S.), which stands 390 m. upstream from the new dam, will be fully submerged by the waters of the reservoir at the very first storage stage. Once completed, the new dam will raise the water level 123 m. above the crest of the old dam. Special attention is paid in the article to the composition of the concrete used in the dam and to the concreting work itself. A series of six cross-sections of the dam at various construction stages from June 6, 1957, till completion at the end of 1961, gives a perfect idea of the constructional programme adopted. An interesting feature is that the keying gaps left between individual blocks of concrete placed from the second stage on are filled during the following spring, when the reservoir is practically empty and conditions are most favourable to the state of constraint obtaining at the time. The water intake for the turbines of the five 37,500 kVA sets of the existing Chandoline plant, as well as the new Fionnay power house (first stage of the Grande Dixence generating plants) at the normal rate of 10 cu. m. and 45 cu. m. per second respectively, is incorporated in the dam itself. This intake can be coupled with the main emptying conduit so as to cope, when required, with an aggregate discharge of up to 105 cu. m. per sec. The Fionnay underground machine hall which, when completed, will house six 60,000 kVA horizontal double-runner Pelton sets, will be supplied by the pressure tunnel 8·58 km. long, 4·1 m. diameter, which was put into operation last July. A restricted-orifice surge tank of 10,000 cu. m. capacity is inserted between the tunnel and the 1,113 m. long reinforced concrete-lined shaft leading to the turbines. This shaft ranges in diameter from 3 m. at the top to 2·8 m. at the bottom, and the lining thickness ranges from 15 to 54 mm. The Fionnay tailrace pours into a 175,000 cu. m. balancing reservoir feeding a pressure tunnel, 16·247 km. long and 4·3 m. in diameter, which connects through a surge tank with a pressure conduit 849 m. long, 3·1 m. diameter, a vertical shaft 166 m. long and 2·9 m. diameter, and an inclined shaft 648 m. long and 2·9 m. diameter. The latter leads to the six 80,000 kVA sets of the underground Nendaz power house built on the left bank of the Rhone. These generating sets are similar in design to the Fionnay sets, but the turbines are 45,000 h.p. units, so that the total installed capacity reaches 540,000 h.p. The number of turbines and their discharge is the same in both plants



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The illustration shows the trial erection of the N'Kata Bay Landing Stage, constructed from sixteen Uniflotes, decked with Storey Steel Decking and connected to land with Bailey Access Bridges. Photo by courtesy of Sir William Halcrow & Partners.

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so as to facilitate load distribution, the plants operating in cascade. The electrical equipment of the two plants is briefly described. At Fionnay, civil-engineering work is practically completed. Two generating sets have been in service since July 1957, and a third was scheduled to go on the line last November. Construction work at the Nendaz installations is in progress. The pressure tunnel has been attacked at four points: Fionnay, Sarreyer, Isérables and Nendaz. Earthwork for the Nendaz pressure conduit is also advancing, and the boring of the vertical shaft has been completed. In the underground power house, the main roof, 19 m. wide by 180 m. long, as well as the secondary roof, has also been completed. The first sets of the Nendaz plant are expected to go on the line about the end of 1959. (Jacques Desmeules, *Le Génie Civil*, Vol. 134, No. 21, November 1, 1957, p. 433, 14 pp., 17 ff.)

Gadmen Valley Intakes

A detailed description is given of the intakes which collect the waters of three streams fed by the Gadmen glacier through a single, 13 km. long tunnel discharging into the main power tunnel of the Innertkirchen power house. A common characteristic of the three intakes, set at altitudes of 1,347·0, 1,340·5 and 1,324·5 m. respectively, is that they are sited in the path of avalanches and become totally inaccessible in winter. Also all three streams have at the intake sharp gradients varying between 8 and 20%, and it was deemed advisable to equip each intake with a small forebay. As in all mountain intakes, solid transportation, mostly sand, presented awkward problems. Economic considerations led to a solution which widely differs from the traditional method and involves utilising the major part of the flow at high-water periods to carry away the silt load. At each intake, settling tanks are provided to collect drifting material, an evacuation of which is then either restricted to high-flood periods, or postponed to periods of minimum energy requirements. The Wenden intake is designed for a maximum capacity of 2·0 cu. m. per sec., the water being retained by a massive 6·2 m. high sill with built-in bottom outlet and a tainter gate 2·0 by 1·0 m. Inlet and sand-trap are both built across the direction of the flow. The inlet has a free cross-section 1·75 m. high by 3·5 m. wide to enable the water to enter it even at slow velocities. The floor of the sand-trap is inclined on both sides against the flushing canal, and a slide gate 80 by 120 cm. enables it to be emptied in the shortest time. In view of the exposed position of the Stein intake, all its constituent parts, i.e. intake proper, desilter, bottom outlet, flushing system and the junction with the Wenden supply conduit, are housed in a massive sill. The inlet, designed for an intake of 7·0 cu. m. per sec., has two 1·3 by 2·5 m. openings fitted with fine trashracks. The desilter is of the same type as at the Trift intake. Transportation conditions are here much more difficult than at the Wenden intake, and the whole structure is designed for a much greater volume of silt. Of all three structures, the Trift intake has to face the most difficult conditions, both as to avalanche hazards and silt load. At the intake site, however, the stream is wide enough to enable the sand-trap to be built separately from the weir and its sill. The mechanical drive of the gates of the bottom outlet and the control system is housed in the sill and requires considerably more room than at the Stein intake. The desilting chamber, which is built in an extension of the sill on the right bank of the stream, has been described, together with the cellular desilter in WATER POWER, April 1958, p. 160. All the gates are electrically controlled from the Innertkirchen power house. (Dipl.-Ing. H. Juillard, *Schweizerische Bauzeitung*, Vol. 76, No. 9, March 1, 1958, p. 119, and No. 10, March 8, 1958, p. 135; 12 pp., 25 ff.)

Operating Experiences with Kemano Turbines

This paper, read before a meeting of the Vancouver Branch of the Engineering Institute of Canada, describes the installation of the first three 150,000 h.p. four-nozzle impulse turbines in the Kemano power station of the Aluminum Company of Canada Limited, and gives operating data and test results. The turbines operate under a head of 2,500 ft. and have a speed of 327 r.p.m. All the turbines, as tested by Norman R. Gibson, the consulting engineers of Niagara Falls, N.Y., showed a high efficiency (90·5% to 91%) between the ranges 90,000 to 150,000 h.p., and all exceeded the stipulated output at maximum load. With two of the runners cavitation occurred within two to four weeks, this taking place at the back of the bucket splitter tips. This problem was tackled by improving the bucket contour in the critical area and by a more satisfactory metallurgical technique.

The objective was to produce an 18–8 alloy tip on the buckets and this was attempted by a straight build-up and then welding on the prefabricated tips, a layer of 25–20 alloy being first laid down on the carbon steel of the bucket. Serious cracks developed and it was concluded that 25–20 alloy was causing the trouble; a change to 29–9 removed this difficulty. Although the contour problem has not been completely solved, a substantial improvement has been made and it has been possible to lengthen the inspection interval very considerably. Excellent results have been obtained with the third runner (13% chrome), the only snag with this material being that it is difficult to weld, needs a preheat of 600°F. and needs to be stress relieved afterwards.

The Kemano water carries no foreign material whatever and all the throat rings, needles and nozzles are in an excellent condition. (J. T. Madill, M.E.I.C., and F. P. Gordon, Jr.E.I.C., *The Engineering Journal*, February 1958, Vol. 41, No. 2, pp. 50–56.)

New E.D.F. Projects

Upper Middle Durance Development. Within the frame of the development of the section of the Durance comprised between Serre-Ponçon and Cadarache, the three applications filed in 1956 for the equipment of the Oraison, Manosque, Sainte-Tulle II and Beaumont heads (aggregate production assessed at 1,000 million kWh) are now at the Public Inquiry stage. This section, which reaches from the Serre-Ponçon tailwater to the backwater of the planned Oraison reservoir, is to be developed in three stages: 1) Curbans (installed capacity 140,000 kW, output 407 million kWh); 2) Sisteron (180,000 kW, 519 million kWh); 3) Aubignosc (58,000 kW, 212 million kWh). The Curbans intake will draw its water supply from the Serre-Ponçon balancing reservoir; the Sisteron plant, with a 33 km. long supply canal as its

main feature, will draw 220 cu. m. per sec. from the Durance and 40 cu. m. per sec. from the Buech to the three turbines of the underground power house, and return the discharge to the Buech. The smaller Aubignosc plant, set up immediately upstream of the Durance-Jabron confluence, will return the discharge from its two turbines into the head pond of the Oraison plant.

Les Bois *(Haute-Savoie)*. This Alpine development, sited about two miles from the town of Chamonix, is to harness the waters of the Arbeyron, a tributary of the Arve, at a head of 379 m., by means of an intake located under the ice of the "Mer de Glace"; the intake works will comprise a diversion tunnel, a desilter, and a flood-control and flushing tunnel. A concrete-lined shaft excavated in the rock will be inserted between the 1,470 m. long supply tunnel, designed for a maximum discharge of 17 cu. m. per sec., and the underground power house. Yearly production is assessed at 107 million kWh.

Besserve (Puy-de-Dôme). This 31,600 kW scheme harnesses the waters of the Sioule, the main left-bank tributary of the Allier river, about 20 miles northwest of Clermont-Ferrand, and will replace the two old plants at Les Fades and Chambonnet whose aggregate output of 37 million kWh it will exceed by 41 million kWh, not to mention another 2 million kWh resulting from the improved working of the Queuille downstream plant. *(La Technique de l'Eau et de l'Assainissement*, Vol. XII, No. 135, March 15, 1958, p. 30.)

Pore Pressure in Earth Dams

This is a comprehensive investigation in three instalments and seven chapters, devoted to the determination of pore pressure in earth dams. The various theories of pore pressure in earth dams are analysed, an endeavour being made to take into account all the studies published on the subject in the course of the last 20 years. Chapter I gives a historical survey of the various applications of earth dams, as well as of construction practice, compacting, hydraulic fill and water-content control being dealt with in separate sections. In Chapter II the various methods applied to the determination of pore pressure in earth dams during their construction are reviewed. It is shown that the present methods are at best approximate since they neglect some of the factors affecting the magnitude of pore pressure. In Chapter III (second instalment), the author deals with the determination of pore pressure under normal permanent operating conditions, which is of particular importance for the stability analysis of the downstream slopes of earth dams. Chapter IV investigates the problem under the conditions brought about by the emptying of the reservoir, a problem of special significance with regard to the stability of the upstream slopes. A distinction is drawn between dams which can be treated as rigid and those which are compressible. In dams of the former category, the influence of capillarity being assumed as negligible and material as saturated, pore pressures can be assessed by means of flow nets. In instances of saturated compressible dams, the increase in pore pressure may be safely assumed as equal to the increase in pressure due to the weight of the dam. The collected measurement results will enable greater economy to be achieved in determining the dimensions of the dam. In the final instalment of the article (chapters V to VII), the methods adopted for measuring pore pressure in the laboratory and in the field are reviewed. In the absence of data on field measurements, the determination of pore pressure in the laboratory is important for the verification of the various theories that have been developed. There are two ways of approaching the measurement of pore pressure in the laboratory: one method consists in introducing a porous pilot head into the sample whereas the other measures pore pressures by means of saturated porous plates at the top or bottom of the test specimen. The development of laboratory technique has made it possible to ascertain the extent of pore pressure in undrained samples subject to an arbitrary state of stresses in a triaxial cell. The measurement of pore pressure in the field enables the engineer to check the stability of the dam and to take corrective measures before serious conditions develop. The advantages and disadvantages of the various pore-pressure measuring implements now in use are discussed, the Casagrande open piezometer, the Bureau of Reclamation closed piezometer and the Maihak electric gauge being considered as particularly reliable. Concluding his study, the author expresses the hope that the results of pore-pressure field measurements will in future be given a wider and more frequent publicity which would pave the way to more accurate methods and greater economy in design and construction. (Guy Le Moigne, ingénieur E.T.P., *Travaux*, Vol. 41, December 1957, p. 615, 8 pp., 6 ff.; Vol. 42, No. 280, February 1958, p. 149, 6 pp., 8 ff.; No. 281, March 1958, p. 200, 6 pp., 8 ff.).

Swiss Statistics

The statistics issued a short time ago by the Federal Electricity Office, Berne, give the production and consumption figures for the hydrographic year 1956/57. The very detailed first chapter deals with Swiss production and consumption as a whole. Compared with the year 1930/31, hydraulic production has increased from 5,026 to 15,704 million kWh, and consumption from 3,856 to 14,653 million kWh. Figures are also given separately for winter and summer production and consumption. (*Bulletin de l'Association Suisse des Electriciens*, Vol. 49, No. 6, March 15, 1958, p. 223, 20 pp., 10 tables, 13 graphs.)

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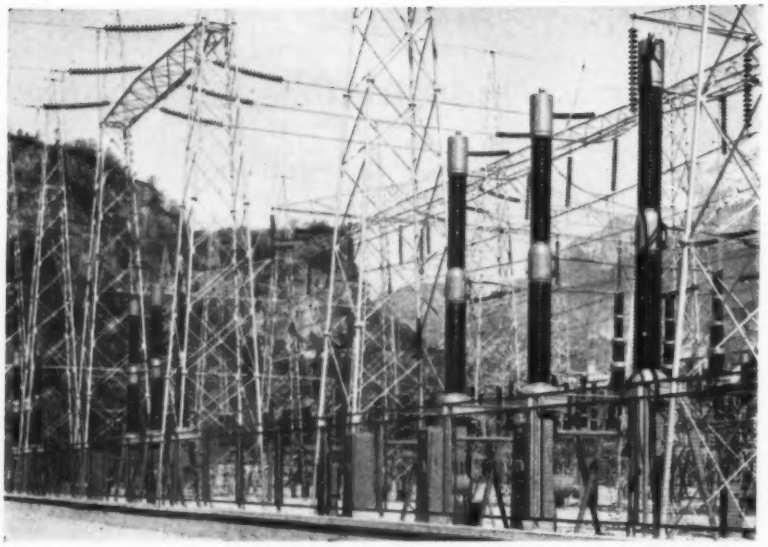

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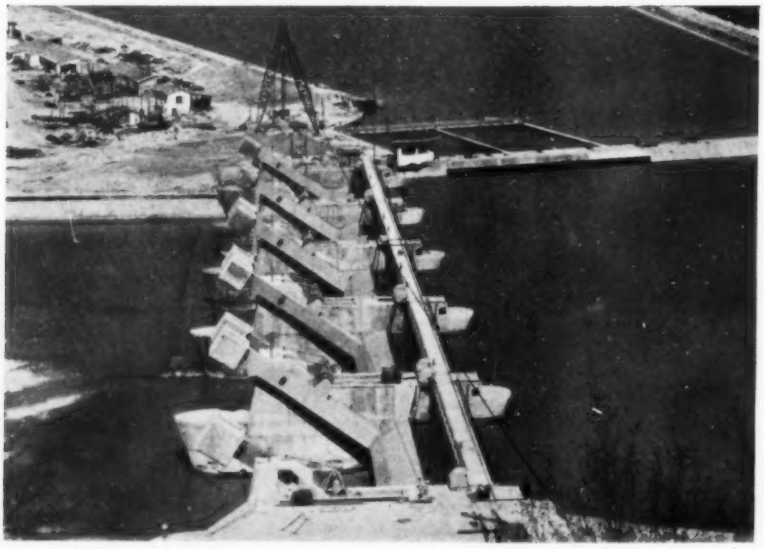

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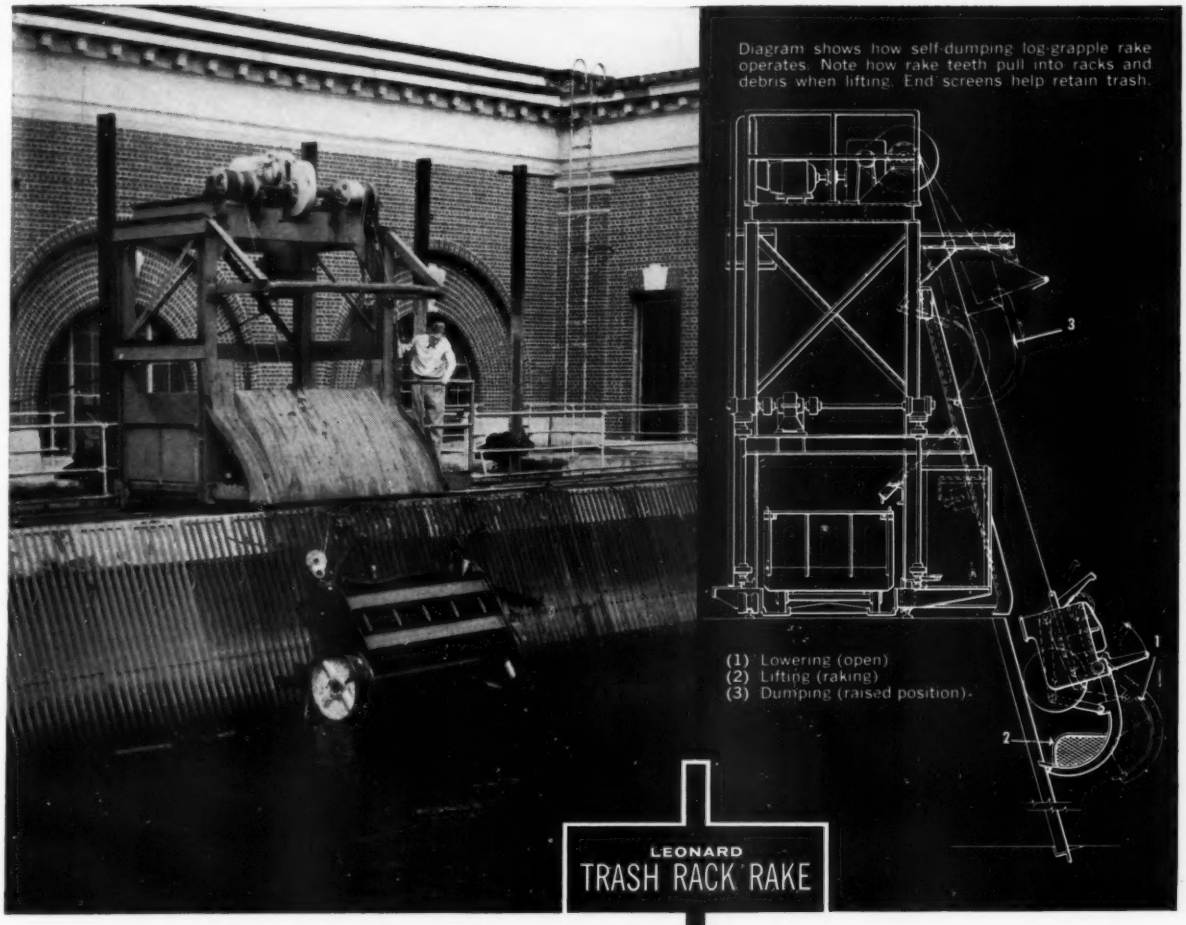
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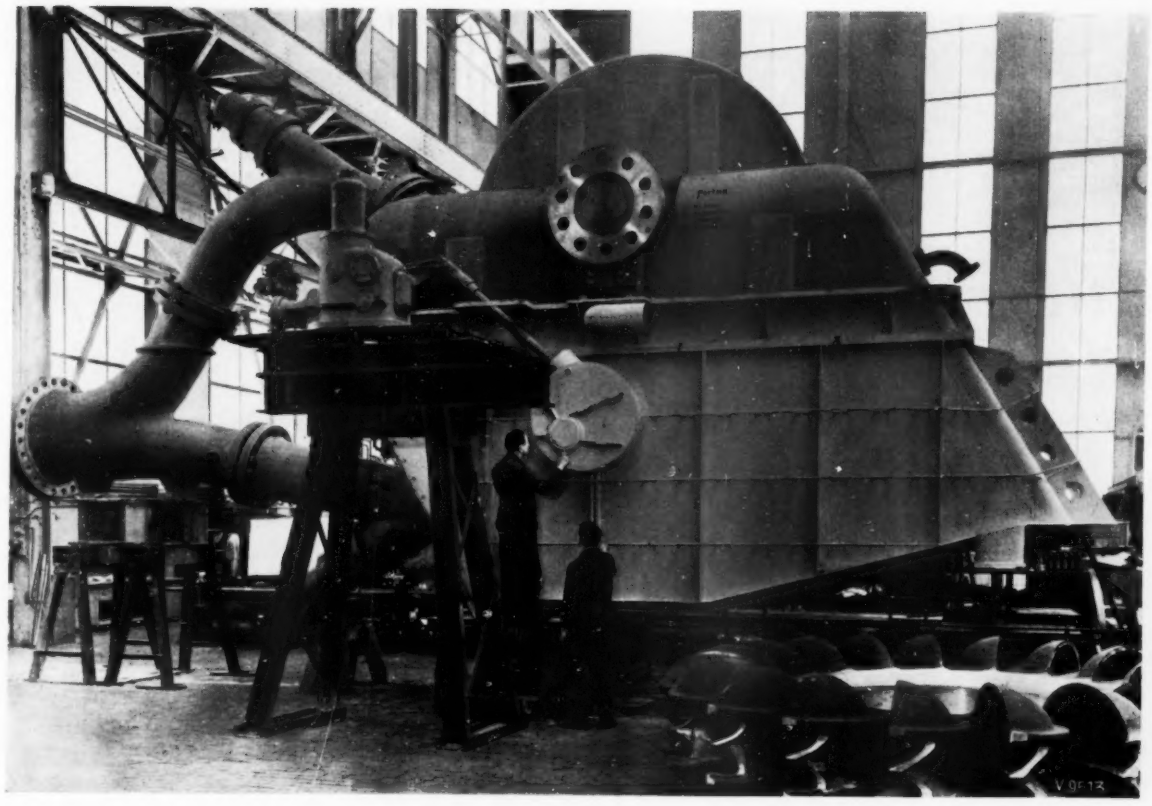

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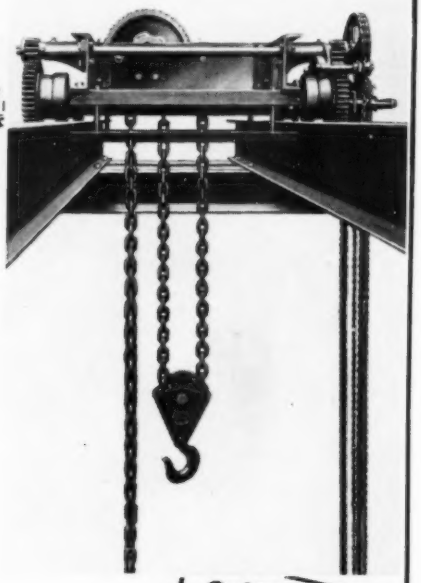

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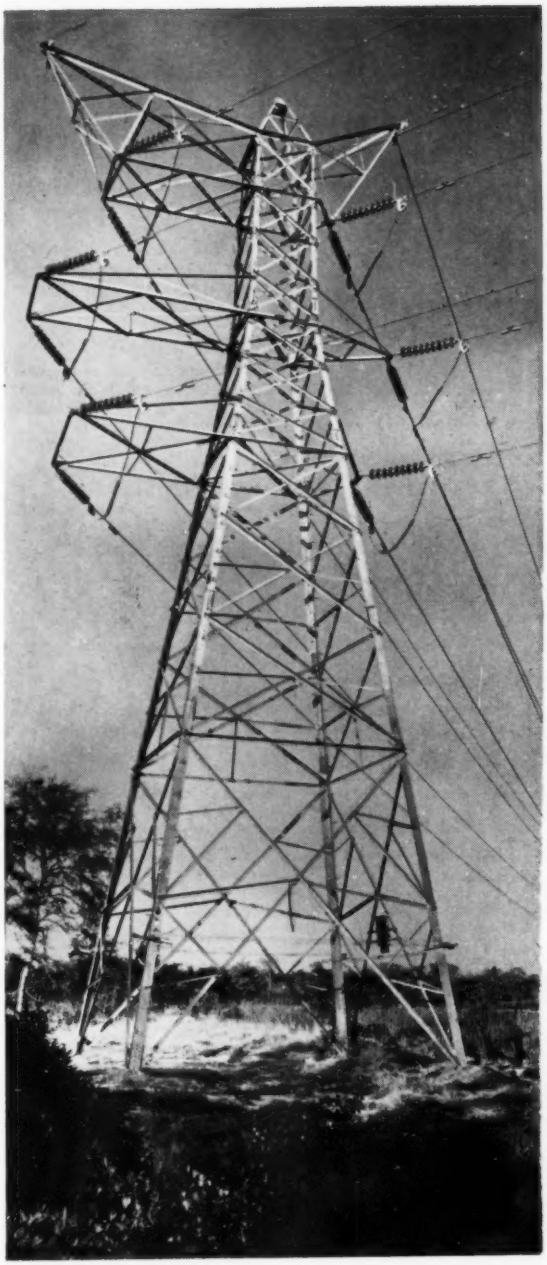

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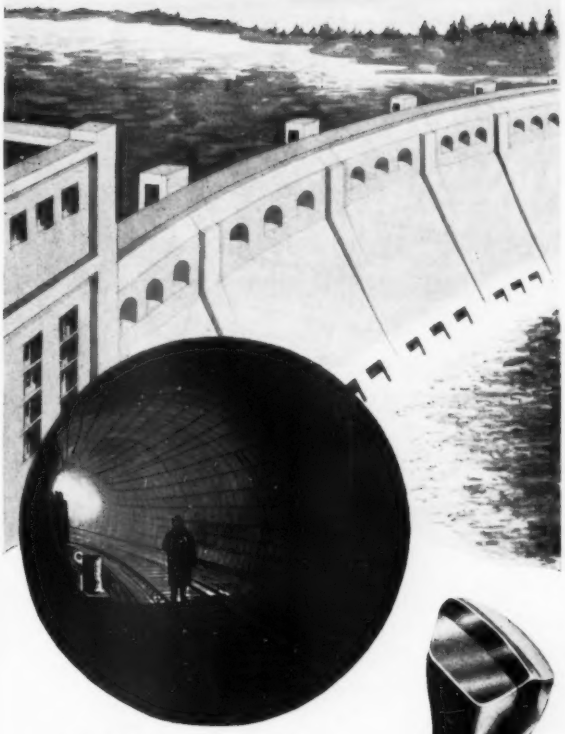

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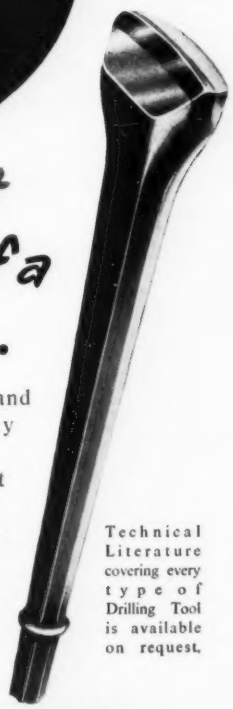

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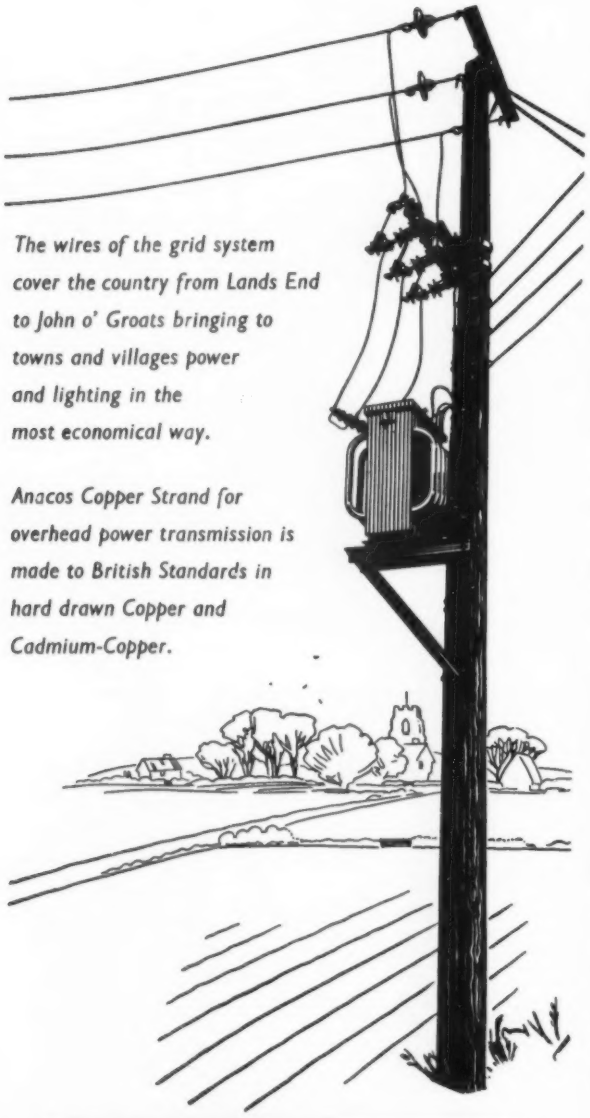

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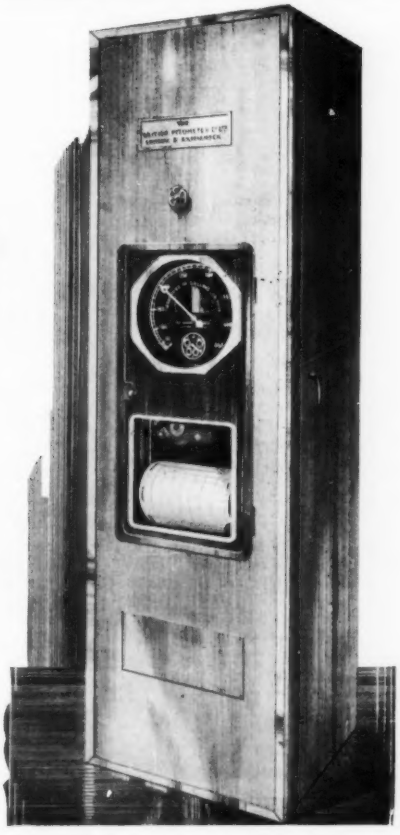

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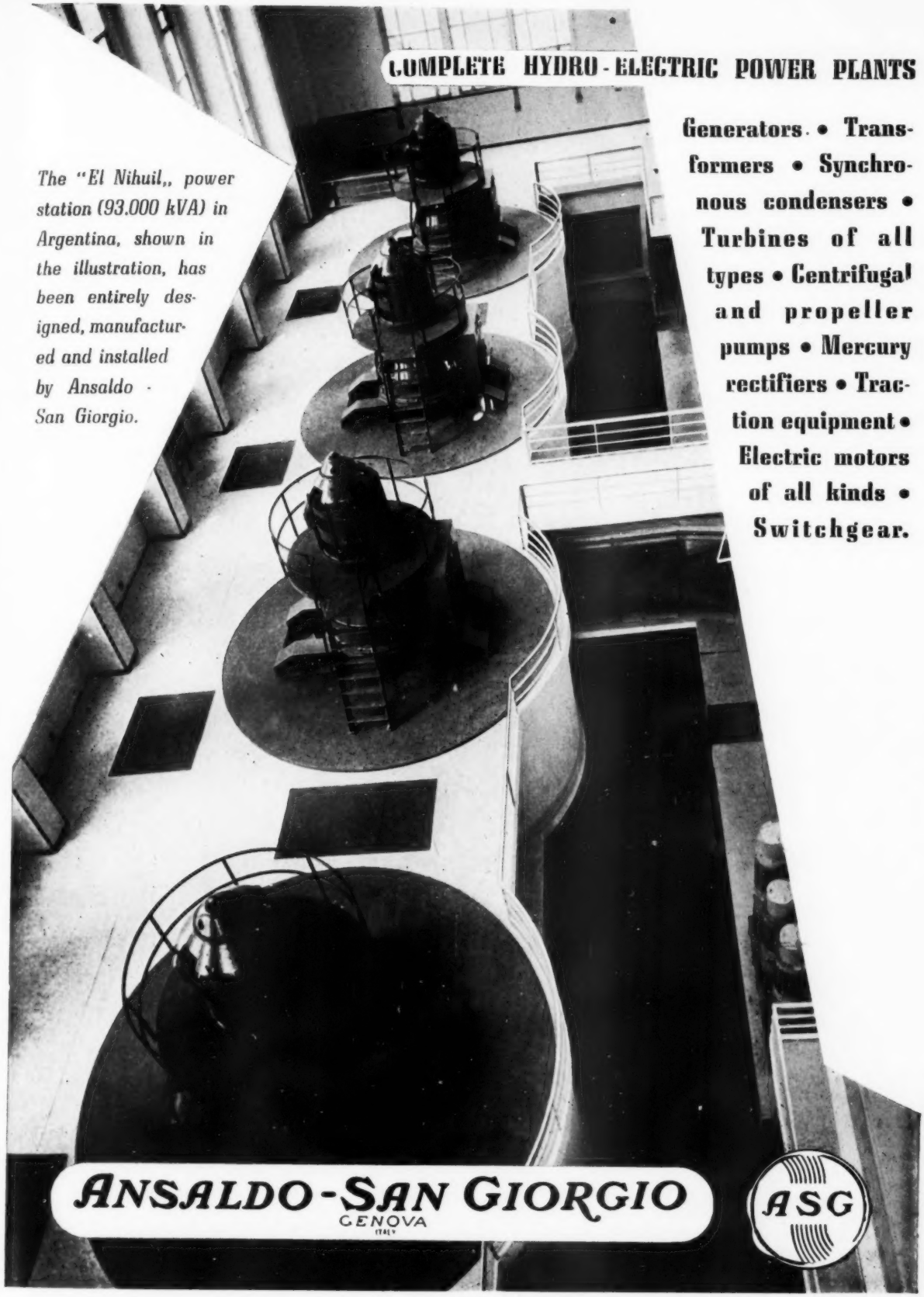

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